AMERICAN CONCRETE INSTITUTE

PROCEEDINGS

OF THE

TWENTY-THIRD ANNUAL CONVENTION

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TECHNICAL COMMITTEES.

(For convention year February, 1927, to February, 1928.)

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G-2.—Advisory.

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G-3.—Form of Standards.

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G-4.—Nomenclature.

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J-2.—INSTITUTE REPRESENTATION ON JOINT CULVERT PIPE COMMITTEE.

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J-3.—TECHNICAL PROBLEMS IN CEMENT MANUFACTURE.

H. C. TURNER, *Chairman.*
M. M. UPSON, W. K. HATT, RICHARD L. HUMPHREY.

J-4.—CEMENT.

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E-6.—Destructive Agents and Protective Treatments.

A. E. Lindau, *Chairman.*
F. R. McMillan, *Secretary.*
W. K. Hatt, George E. Beggs, R. B. Young, G. F. Loughlin, P. J. Freeman, E. Viens.

E-7.—Waterproofing.

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E. H. Berger, *Secretary.*
Maximilian Toch, Duff A. Abrams, J. C. Pearson, Herman Fougner, W. A. Frerer, S. R. Church, M. O. Withey, Joseph Rose, J. H. Libberton, Ernest Ashton, G. L. Lucas, Raymond H. Krecker, C. N. Forrest, A. W. Stephens, A. E. Horn, A. B. Cohen, Ralph Atwater, F. M. Dawson.

C-2.—Concrete Floor Finish.

E. E. Davis, *Chairman.*
H. S. Wright, *Secretary.*
J. E. Freeman, W. M. Rynerson, A. C. Tozzer, H. R. Wood, T. A. Pearson.

C-3.—Treatment of Concrete Surfaces.

W. E. Hart, *Chairman.*
F. A. Hitchcock, *Secretary.*
J. C. Pearson, J. G. Ahlers, E. D. Boyer, Cloyd M. Chapman, Wharton Clay, J. E. Freeman, H. A. Gardner, H. Whittemore Brown, Napoleon M. Bernier, Samuel Warren, O. A. Malone, W. H. Shaffer, Jr., J. Miller Porter.

C-4.—Forms for Concrete Building Construction.

E. C. Harding, *Chairman.*
D. B. Neiderlander, R. E. Bates, W. J. Camlin.

C-6.—Field Methods.

R. B. Young, *Chairman.*
F. R. McMillan, C. E. Nichols, John G. Ahlers, J. B. Hunley, W. J. Krefeld.

S-1.—Reinforced-Concrete Chimneys.

C. E. Nichols, *Chairman.*
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S-2.—CONCRETE BRIDGES.

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P-4.—CONCRETE STAVES.

W. O. BRASSERT, *Chairman.*
W. G. KAISER, *Secretary.*
C. C. WOODY, KENT MERRY, S. E. ANDERSON.

P-6.—CONCRETE PRODUCTS PLANT OPERATION.

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P-7.—PIPE, DRAIN TILE, AND CONDUIT.

C. F. BUENTE, *Chairman.*
M. W. LOVING, *Secretary.*
ERNEST F. BENT, M. L. ENGER, J. J. LONGLEY, R. S. LANDER, J. A. DUNN, J. S. LAMBIE, C. H. BULLEN, SAMUEL FORTIER, PAUL KIRCHER, B. S. PEASE, WILLIS WHITED, E. S. RANKIN, R. A. FOLEY.

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BY-LAWS.

AMERICAN CONCRETE INSTITUTE.

ARTICLE I.

MEMBERS.

SECTION 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to co-operate in the purposes of the Institute, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

SEC. 2. A firm or company shall be treated as a single member.

SEC. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

SEC. 4. Application for membership shall be made to the Secretary on a form prescribed by the Board of Direction. The Secretary shall submit monthly or oftener, if necessary, to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at the time with a statement of the qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

SEC. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.*

ARTICLE II.

OFFICERS.

SECTION 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the

* NOTE.—There are before the members of the Institute for ratification by letter ballot two amendments to these by-laws: (1) To Section 2, Article IV, increasing the annual dues to $12.50 for membership years beginning on or after July 1, 1927; (2) Providing a new Section 6, Article I, empowering the Board of Direction to elect Honorary Members. Both amendments were adopted at the 23rd annual convention Feb. 24, 1927. The letter ballot was canvassed May 26, 1927.

Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction.

SEC. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

SEC. 3. There shall be a Committee of five members on Nomination of Officers elected by letter ballot of the members of the Institute, which is to be canvassed by the Board of Direction on or before September 1 of each year.

The Committee on Nomination of Officers shall select by letter ballot of its members, candidates for the various offices to become vacant at the next Annual Convention and report the result to the Board of Direction who shall transmit the same to the members of the Institute at least 60 days prior to the Annual Convention. Upon petition signed by at least ten members, additional nominations may be made within 20 days thereafter. The consent of all candidates must be obtained before nomination. The complete list of candidates thus nominated shall be submitted 30 days before the Annual Convention to the members of the Institute for letter-ballot, to be canvassed at 12 o'clock noon on the second day of the Convention and the result shall be announced the next day at a business session.

SEC. 4. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-President and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election as President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In case of the disability or neglect in the performance of his duty of any officer of the Institute, the Board of Direction shall have power to declare the office vacant. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

SEC. 5. The Board of Direction shall have general supervision of the affairs of the Institute and at the first meeting following its election,

appoint a Secretary and from its own members a Finance Committee of three; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the Institute, and shall appoint a chairman for each committee. Four or more additional members on each special committee shall be appointed by the President, in consultation with the Chairman.

SEC. 6. It shall be the duty of the Finance Committee to prepare the annual budget and to pass on proposed expenditures before their submission to the Board of Direction. The accounts of the Secretary and Treasurer shall be audited annually.

SEC. 7. The Board of Direction shall appoint a Committee on Resolutions, to be announced by the President on the first regular session of the Annual Convention.

SEC. 8. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

SEC. 9. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

SEC. 10. The President shall perform the usual duties of the office. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

SEC. 11. The Secretary shall be the general business agent of the Institute, shall perform such duties and furnish such bond as may be determined by the Board of Direction.

SEC. 12. The Treasurer shall be the custodian of the funds of the Institute, shall disburse the same in the manner prescribed and shall furnish bond in such sum as the Board of Direction may determine.

SEC. 13. The Secretary shall receive such salary as may be fixed by the Board of Direction.

ARTICLE III.

MEETINGS.

SECTION 1. The Institute shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days prior to the date of Convention.

SEC. 2. The Board of Direction shall meet during the Convention at which it is elected, effect organization and transact such business as may be necessary.

SEC. 3. The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

SEC. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction of the Executive Committee.

ARTICLE IV.

DUES.

SECTION 1. The fiscal year shall commence July 1st.

SEC. 2. The annual dues shall be ten dollars ($10.00) payable annually in advance from first of the month following notification of the applicant of his election by the Board of Direction.*

SEC. 3. Each member shall be entitled to receive one copy of one volume of the *Proceedings* for each membership year and additional volumes at a price fixed by the Board of Direction.

SEC. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon payment of all indebtedness against them upon the books of the Institute.

ARTICLE V.

STANDARDS.

SECTION 1. Proposed new or revised Standard Specifications, Standard Practice, and Standard Definitions when approved by a majority voting in the committee in which they originate, shall be submitted, in the form adopted in the Standard Form of Standards, to the Secretary of the Institute sixty days prior to the opening of the Annual Convention at which they are to be presented. The Secretary of the Institute shall cause these proposed new standards or revised standards to be printed as Proposed Tentative Standards and mailed to the full membership of the Institute thirty days prior to the opening of the convention. As there amended and approved, they shall be published in the Annual *Proceedings*, next issued as Tentative Standards. At a subsequent Annual Convention, they may again be offered unamended, by their originating committees as proposed standards, and as there approved by a majority of those voting, they shall be submitted to letter ballot of the Institute membership, to be canvassed within ninety days thereafter. Such proposed standards shall be considered adopted unless at least 10 per cent of those voting shall vote in the negative.

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ARTICLE VI.

AMENDMENTS.

SECTION 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

SUMMARY OF PROCEEDINGS OF THE TWENTY-THIRD ANNUAL CONVENTION.

First Session, Tuesday, February 22, 1927, 2 P. M.

President Upson in the chair.

The convention was called to order by Maxwell M. Upson, president of the American Concrete Institute who presented an address reviewing the progress in knowledge of and practice with concrete and its economic importance.

A paper by Arthur R. Lord, president, Lord and Holinger, engineers, Chicago, "Notes on Concrete—Wacker Drive, Chicago," was formally presented but not read, having been preprinted and distributed in advance of the convention. Mr. Lord confined himself to an informal talk on outstanding features of the work in the course of which he presented motion pictures.

A paper, "Construction of the Bremerton Pier, Navy Yard, Bremerton, Washington," by Lt. Comdr. J. J. Manning (C. E. C.), U. S. Navy, was read in the author's absence by Lt. Comdr. H. F. Bruns (C. E. C.), U. S. Navy.

R. B. Young, chairman, Committee C-6, Field Methods, presented the committee's report: "Economics of Field Control of the Quality of Concrete"; the report having been preprinted was presented in abstract.

E. C. Harding, chairman, Committee C-4, Forms for Concrete Building Construction, presented the committee's report, "Steel Forms for Flat Slab Construction."

C. R. Ege, secretary, Committee S-6, Concrete Roads and Pavements, presented the report of the committee consisting of "Proposed Specifications for Concrete Curb and Concrete Curb and Gutter" to replace the Institute's old specifications on the subject. The report (preprinted) was adopted as a tentative standard of the Institute.

Second Session, Tuesday, February 22, 1927, 8 P. M.

Past President W. K. Hatt in the chair.

F. R. McMillan, chairman, Committee E-1, Reinforced Concrete Building Design and Specifications, presented the report of the committee (preprinted), "Tentative Building Regulations for the Use of Reinforced Concrete" (E1-A-27T). On motion of the chairman of the committee the report was amended in Chapter B, Definition "Aggregate" so that this paragraph now reads:

> Aggregate.—Inert material which is mixed with portland cement and water to produce concrete; in general, aggregate consists of sand, pebbles, gravel, crushed stone, blast furnace slag or similar materials.

The amendment also provided that the specifications include a note defining blast furnace slag as in the 1925 report of Committee E-5, Aggregates, p. 524, *Proceedings*, American Concrete Institute, Vol. 21, as follows:

> Blast-Furnace Slag.—The non-metallic product, consisting essentially of silicates and alumino silicates of lime, which is developed simultaneously with iron in a blast furnace.

This report on the definition was made by Committee E-5 in its comment on a definition in the Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete.

Another amendment was offered, but lost, to remove the limitation on use of rail steel bars of the larger sizes.

A third amendment is to Chapter C, Materials and Tests, paragraph C-7, Metal Reinforcement, to include a tentative specification of the American Society for Testing Materials on wire reinforcement. The amendment was adopted.

Subject to the two amendments admitting "blast-furnace slag" and "wire reinforcement" the specifications as preprinted were adopted as tentative.

A paper, "The Design and Construction of the Grandfey Viaduct," by Adolph Buhler, chief engineer of bridges, National Swiss Rys., was presented in the author's absence by Prof. George E. Beggs, Princeton University.

Albert Smith, Smith & Brown, consulting engineers, Chicago, presented a paper, "Design of Concrete Buildings for Wind Stresses."

The paper, "Report of Tests on 300-ft. Reinforced-Concrete Chimney," which had been preprinted was presented in brief abstract by the author, Earle D. McKay, Universal Portland Cement Co.

"The Reinforced-Concrete Column," a paper by Phil. J. Markmann, consulting engineer, St. Louis, Mo., which had been preprinted was presented in brief abstract.

A sudden change in the plans of M. Abe, consulting engineer, Tokyo, Japan, visiting in this country, prevented his attendance at the convention and the presentation of his announced paper, "Reinforced-Concrete Structures in Japanese Earthquake."

THIRD SESSION, WEDNESDAY, FEBRUARY 23, 1927, 9.30 A. M.

Vice-President E. D. Boyer in the chair.

C. L. Bourne, secretary, presented the report of Committee P-1, Standard Concrete Building Units, in four parts: (1) "Proposed Specifications for Concrete Manhole and Catch Basin Block," which the committee offered as tentative and which was thus adopted; (2) "Tentative Specifications for Concrete Building Block and Concrete Building Tile," proposed for final adoption by including amendments presented in 1925 and 1926, to the previous standard specifications of 1925. P1-A-25. (This was adopted by the

convention and referred to letter ballot); (3) "Tentative Specifications for Concrete Brick" containing 1925 and 1926 tentative amendments to the 1925 standard specifications (P1-B-25—preprinted with amendments P1-B-26T) which the committee recommended for final adoption. The convention voted to continue these as "tentative" another year in the hope that co-operation with a committee of the American Society for Testing Materials may result in the adoption of a single standard for brick; (4) "Tests on Concrete Building Tile" was accepted by the convention for publication subject to the insertion of a note explaining why in dry-tamp mixtures made on the so-called dry side of the cement water ratio curve, strength decreases with less water, rather than increases with less water, as in general construction with wetter mixtures.

Benjamin Wilk, chairman, and Earle D. McKay, member, Committee P-6, Concrete Products Plant Operation, presented its report, "Tests of Outdoor Curing of Concrete Products." Then followed a symposium of brief papers on "Operating Problems in Concrete Products Manufacture," with papers by the following:

W. R. Warford, Warford Construction Co., Aurora, Ill.; George Chandler, Superior Products Co., Detroit, Mich.; Austin Crabbs, Cement Products Co., Davenport, Ia.; C. J. Herzog, General Cement Products Co., Pittsburgh, Pa.; Benjamin Wilk, Standard Building Products Co., Detroit, Mich.; Newton D. Benson, Providence, R. I.; John A. Goetz, Mattoon, Ill., whose paper was presented by title only; and Lacy Peyton, Benton, Ill., whose paper was presented in the author's absence by Benjamin Wilk.

E. Y. Bragger and W. F. Lockhardt were appointed by the chair as tellers on ballots cast for officers of the Institute.

Two hundred and forty-three Institute members and friends sat down to the informal luncheon in the foyer of the grand ball room at noon, Feb. 23, luncheon having been arranged by the greetings committee, John G. Ahlers, chairman. Francis J. Straub, who was on the Keith vaudeville circuit before he gave the major portion of his time to cinder concrete building units, entertained the crowd with sleight-of-hand.

FOURTH SESSION, WEDNESDAY, FEBRUARY 23, 1927, 2 P. M.

Vice-President John J. Earley in the chair.

W. Paul Eddy, treasurer, Onondaga Litholite Co., Syracuse, N. Y., presented the paper, "Cut Cast Stone Manufacture," by Henry P. Warner, president of the Onondaga Litholite Co. who was unable to be present.

Robert F. Havlik, Havlik Stone Co., Aurora, Ill., presented a paper, "Why I Make Wet Cast Stone," which he did not read. Instead, he conducted a demonstration of acid washing and of glue and plaster mold work.

"How We Make Art Marble" was the title of a paper presented by I. L. Stearn, Chicago Art Marble Co., Chicago.

"Color Pigments for Use in Concrete" was presented by Raymond Wilson, Portland Cement Association research laboratory.

FIFTH SESSION, WEDNESDAY, FEBRUARY 23, 1927, 8 P. M.

President Upson in the chair.

Frederick Lee Ackerman, architect, New York, presented a paper, "A Preliminary Note on Concrete Design—The Architectural Theory."

A paper, "The Use of Concrete on the Pacific Coast—Its Structural and Aesthetic Possibility," by John C. Austin, architect, Los Angeles, was presented in the author's absence by Richard Powers, architect, Chicago.

Irving K. Pond, architect, Chicago, presented a paper, "Architectural Uses of Concrete."

A paper entitled, "The Behavior of Engineering Structures in Recent Violent Wind Storms," by Roy L. Peck and Norman M. Stineman, Portland Cement Association, Chicago, was presented for publication in the *Proceedings*, the authors presenting stereopticon views of damage by the storm with brief descriptive notes of the pictures shown.

SIXTH SESSION, THURSDAY, FEBRUARY 24, 1927, 9.30 A. M.

Past-President A. E. Lindau in the chair.

F. H. Jackson, secretary of Committee E-5, Aggregates, made formal presentation of the report of the committee which had been preprinted, consisting of reports of sub-committees on Gravel, Crushed Stone and Slag.

G. F. Loughlin, geologist, U. S. Geological Survey, Washington, D. C., presented his paper, "Qualifications of Different Kinds of Natural Stone for Concrete Aggregates."

R. H. Bogue, research director, Portland Cement Association fellowship at the United States Bureau of Standards, presented a paper, "Portland Cement in Concrete Engineering," describing the purpose and character of a fundamental investigation of portland cement.

The report of Committee E-3, Research, H. F. Gonnerman, chairman, on "Researches on Concrete Materials and on Plain and Reinforced Concrete" was preprinted and presented by title only.

Herbert J. Gilkey, associate professor of civil engineering, University of Colorado, presented a brief abstract of his preprinted paper, "The Coarse Aggregate in the Concrete as a Field for Needed Research."

"A New Test for Workability of Concrete," by Prof. Tokujiro Yoshida of Kyushu Imperial University, Japan, was presented by F. E. Richart, research assistant professor of theoretical and applied mechanics, University of Illinois.

SEVENTH SESSION, THURSDAY, FEBRUARY 24, 1927, 2 P. M.

President Upson in the chair.

There was first presented the report of the tellers announcing the election of the following officers and directors:

President—Maxwell M. Upson, term, one year.

Vice-President—Edward D. Boyer, term, two years.

Treasurer—Harvey Whipple, term, one year.
Directors, 3rd district—S. C. Hollister,
4th district—P. H. Bates,
5th district—D. A. Abrams, each for a term of two years.

Two proposed amendments to the By-Laws were unanimously adopted by the convention and referred to letter ballot of the membership:

First, to Article IV, Section 2.—Section 2 to be changed to read as follows:

Section 2. The annual dues shall be twelve dollars and fifty cents ($12.50) payable annually in advance from the first of the month following notification of the applicant of his election by the Board of Direction.

Second, adding a Section 6 to Article I of the By-Laws as follows:

Section 6. The Board of Direction may confer honorary memberships in recognition of services of an extraordinarily meritorious character before the Institute. Honorary members shall be entitled to full membership privileges without the payment of dues.

The secretary made an announcement in regard to the honor roll, on which are listed those members of the Institute who in the period, Feb. 1, 1926, to Jan. 31, 1927, sponsored three or more new members. Prizes were awarded to the ten on the honor roll who sponsored the most new members—the first prize to Duff A. Abrams, 44 new members; S. Stepanian, 22; F. R. McMillan, 19; M. M. Upson, W. F. Way, 17; Shu-t'ien Li, 15; H. Frank Machner, 13; F. E. Richart and R. F. Egelhoff, 11; W. F. Lockhardt, 10. The first prize was a humidor; the second, a desk fountain pen and stand; third, a leather portfolio; fourth, a gavel; fifth to tenth prizes, fountain pen and pencil sets.

The short business was followed by the conideration of the convention's special theme: "Time as a Factor in Making Concrete." The subject was introduced and outlined by President Upson. Then followed these papers:

"Why Time is a Factor in the Study and Use of Cement," by P. H. Bates, U. S. Bureau of Standards, Washington; "Water-Cement Ratio as a Basis of Concrete Quality," by Duff A. Abrams. Mr. Abrams' paper was presented in his absence by H. F. Gonnerman; "Time as a Factor in Making Concrete Pavement," by T. H. Johnson, Sioux City, Ia.

Most of the discussion was postponed until the evening session when the consideration of the theme was continued.

There was an informal dinner at 6.30 P. M. attended by more than 180. Immediately following the dinner President Upson presented the Wason Medal to A. Burton Cohen, consulting engineer, New York City, for his paper, "Correlated Considerations in the Design and Construction of Concrete Bridges," judged the most meritorious paper presented at the 1926 convention. There followed an address, "Changing Ideals of American Patriotism," by Prof. Percy H. Boynton, University of Chicago.

EIGHTH SESSION, THURSDAY, FEBRUARY 24, 1927, 8.30 P. M.

President Upson in the chair.

This session continued consideration of the theme, "Time as a Factor in Making Concrete," with four papers presenting the experience and the practice in different applications of concrete:

"In Stucco," by John J. Earley, architectural sculptor, Washington, D. C.; "In Concrete Cast in Sand Molds," by W. Paul Eddy, Onondaga Litholite Co., Syracuse, N. Y.; "In Pipe Made by Centrifugal Process," by F. F. Longley, Lock Joint Pipe Co., Ampere, N. J., Mr. Longley's paper being presented in his absence by Ernest F. Bent, Los Angeles; and "In Dry-Tamp Products Manufacture," by George Chandler, Superior Products Co., Detroit, Mich.

THE WASON MEDAL.

AWARDED EACH YEAR TO THE AUTHOR OF THE MOST MERITORIOUS PAPER PRESENTED TO THE PREVIOUS ANNUAL CONVENTION.

Awarded 1927 to

A. BURTON COHEN, for paper, "Correlated Considerations in the Design and Construction of Concrete Bridges," presented to the 1926 Convention.

PREVIOUS AWARDS.

1916 Convention Paper—A. B. MCDANIEL, "Influence of Temperature on the Strength of Concrete."

1917 Convention Paper—CHARLES R. GOW, "History and Present Status of the Concrete Pile Industry."

1918 Convention Paper—DUFF A. ABRAMS, "Effect of Time of Mixing on the Strength and Wear of Concrete."

1919 Convention Paper—W. A. SLATER, "Structural Laboratory Investigations in Reinforced Concrete Made by Concrete Ship Section, Emergency Fleet Corporation."

1920 Convention Paper—W. A. HULL, "Fire Tests of Concrete Columns."

1921 Convention Paper—H. M. WESTERGAARD, "Moments and Stresses in Slabs."

1922 Convention Paper—GEORGE E. BEGGS, "An Accurate Mechanical Solution of Statically Indeterminate Structures by Use of Paper Models and Special Gages."

1923 Convention Paper—J. J. EARLEY, "Building the Fountain of Time."

1924 Convention Paper—RICHARD L. HUMPHREY, for two papers, "Twenty Years of Concrete" and "The Promise of Future Development."

1925 Convention Paper—E. A. DOCKSTADER, for paper, "Reports of Tests Made to Determine Temperatures in Reinforced-Concrete Chimney Shells."

M. M. Upson

President's Address.

By M. M. Upson.*

The wheels of progress turn slowly. To us, who assemble each year to lay on the altar of advancement our gifts of knowledge, the pyre seems to grow in lagging increments. The transition from groping uncertainty to definite knowledge is so slow that we seldom know when we pass from the dim dawn to the noon day.

It is for this reason that it is well at times to stop to take reckoning. Every prudent mariner wants to know not only where he is, but also the progress that he is making toward his destination. To the casual passenger, with his eye on the distant horizon, there is no evidence that the ship has moved; but when the captain shows his noonday chart with the plottings of each day, courage comes to those who are bearing the labor of the voyage.

It may be well for us to take our sextant, not only to find where we are, but also to plot whence we came and whither we are bound. The craft we are sailing is a mighty one. Not only has it proved its seaworthiness through the tests of many a storm, but it has shown that it has power to go forward through untold adverse currents and headon winds. We are proud of it, and are sure to take courage in reviewing its accomplishments.

In our short twenty-two years of existence the annual consumption of cement has increased from 22,000,000 to 170,000,000 bbl. annually. One hundred and seventy million does not register in our imagination, but if we consider that each barrel and one-half probably represents a $15-yd. of in-place concrete, we can see that the value of the ultimate product approaches $1,007,000,000 per year.

Standing up these figures against the annual output of 45,000,000 tons of steel at $45 per ton, it is evident that the two great structural giants of industry are approaching each other in size. One gloats in the aristocracy of many generations; the other is a youth having just received his majority, with an ancestral background which is best left undisturbed.

So it may be seen that we are treating with no infant industry. It merits our continuous and concentrated attention. Its prodigious growth warrants analysis and a candid appraisal of our part in it. A doting mother may occasionally, in the bosom of her family, recount the virtues of her offspring. So I trust we may be forgiven for recounting some of our vicissitudes and successes.

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* President, American Concrete Institute.

Almost twenty-four years ago this month, Mr. Westinghouse called me into his office and asked me if it would be possible to build out of this new reinforced concrete a group of factory buildings which I was then planning for him. I said we would try. Thereafter followed the construction of a series of beams, slabs and girders, which were all tested to destruction. From these tests our formulas of design were derived. Sixteen thousand-pound steel, 550-lb. maximum fiber compression on concrete, with various and wondrous provisions for shear, were the bases of design. The buildings grew, and have thus far stood, but modern design and construction would now blush to own its early progenitor.

Not a year since that date has passed without a marked step forward in the development of the industry; and practically all of these have found their way to the public through the medium of our *Proceedings*.

These *Proceedings* form the guide book of our advance. Every student of progress consults it to get his bearings before he starts on his own expedition. He assembles and assimilates the progress and errors of others so that he may profit by their experience. Thus have we been helping to direct individual and scattered effort into the ever-flowing stream of progress. The progress may seem slow indeed when measured in a single year, but not so when we consider what twenty-two years have done.

About twelve years ago the chief engineer of one of our greatest railroads told me that never in his or my lifetime would a yard of reinforced concrete be used on the right-of-way of his railroad. Within five years it was a standard of his construction.

The building code of every city on this continent has been rewritten, and very largely because it was necessary to introduce this new giant of industry. Outside of high office buildings, the standard of our skeleton construction is of reinforced concrete. So it is as we touch every industry. It has traveled from footings and retaining walls to dams, buildings, bridges, trestles, roads, piles, poles, pipe, piers, silos, elevators, water tanks, and even to coffins—the last resting place of our mortal bodies. Truly it is a cement age!

In these days, when economics is king, no industry thrives unless it pays the price of its popularity in the coin of better product and reduced cost. It is with this coin that our child has bought its way from obscurity to the forefront of popularity. This is the foundation on which its progress rests. Without these ideals ever before us, the permanency of our great structure may well be questioned. It must be the very creed of our faith, the "Ten Commandments on which are wrought the laws and the prophets."

Since accurate statistics are not available it is quite impossible to measure our advancement in definite figures. Nevertheless, it is illuminating and educational to make approximate assumptions from the data that do exist. How far these figures may vary from the actual is of small moment. They stand as giants in our industrial forest, picturesque and stately, whether or no they top their surroundings.

It is estimated by the cement manufacturers that in the year 1925

approximately one-quarter of their product was used in the construction of buildings. This means that in one year 40,000,000 bbl. of cement found their way into buildings in this country.

If we allow 1½ bbl. to each yard of concrete, and consider that the in-place cost of concrete and steel is $30 per yard, the total concrete construction cost aggregates more than $750,000,000.

Since the average cost of the reinforced concrete of a building is usually more than 40 per cent of its total cost, it is apparent that more than $1,500,000,000 of the annual building program is based on our industry. This is no insignificant amount, when consideration is given to the fact that our present $6,000,000,000 annual program of construction carries with it about $2,000,000,000 of construction other than building.

These figures show that concrete is the chief constituent in more than one-third of the value of our annual building program. To have brought this about in slightly more than twenty years is indeed a marked accomplishment.

The magnitude of any industry is always interesting, but unless it is possible clearly to analyze the principles on which great growth is based, further progress is endangered. Let us consider what this building of concrete has saved the owners:

A typical loft building usually can be built out of concrete at a saving of from 15 to 20 per cent over steel construction. As equipment and exterior trim increases this percentage naturally decreases. Assuming an average of 12½ per cent saving, we find that this one-eighth of the $1,500,000,000 program of 1925 equals almost $200,000,000 of economic saving—enough to pay the interest on more than one-quarter of the war debt.

And this is for only one year. Assuming a growth of from nothing twenty years ago to the present day, these figures grow to a gigantic total saving of over $2,000,000,000—a sum equal to one-tenth of the total cost to our government of the World War.

And this is not the only contribution that the industry has put into the pockets of the public. The fire loss in the United States amounts to about $560,000,000 annually. This is slightly more than 0.4 per cent of the $140,000,000,000 of the total insurance written by all the companies in this country. Based on the foregoing figures, the approximate value of the concrete construction at the end of the year 1925 is $15,000,000,000. Fourtenths of one per cent of this fifteen billion shows an annual saving of $60,000,000.

And this takes no credit for the saving of the contents of the buildings, which are considered to have greater value than the buildings themselves.

There is another humanitarian aspect that should be given consideration:

In our country, during the year 1925, 15,000 souls came to their end through the medium of fire. For the purpose of our computations we may assume the loss of life was proportionate to the monetary loss. It is esti-

mated that the burnable value of insurable property in this country is between $300,000,000,000 and $325,000,000,000. Since the fifteen billions of concrete building value undoubtedly house an equivalent value of goods and equipment, we may count the saving as 30/300 or one-tenth of the 15,000 lives. This means a saving of 1,500 lives annually—surely a worthy service to humanity.

Let us give our attention to another influence of far-reaching importance. Certain essential building materials are limited in their supply. They must be conserved to meet our ever-increasing demand. Consider for a moment how concrete has affected the consumption of lumber. From 1899 to 1907 the annual production of lumber in this country was slightly greater than it was from the years 1921 to 1925. In other words, in a building program of eight times the monetary value, or four times the cubic value (we must measure in 1900 costs) there has been an actual decrease in the amount of lumber used. These are startling figures, and, in the writer's opinion, are due almost entirely to the advent of reinforced concrete.

The very foundations of our building economics are affected by this so-called infant industry of ours. Who can say what the price of lumber would be today had the consumption per cubic unit of construction remained at the rate that existed in the year 1900? As it is, the price has more than doubled.

Another industry has been revolutionized through the advent of our much-discussed product. Approximately one-fourth of the cement produced in the year 1925 was used for road construction; that is, 43,000,000 bbl. This has come about through marked improvement in the method of construction. We all recall the grief that was borne by the pioneer builder of concrete highways.

The favor with which this material is being adopted is indicated by the 1925 statistics. In these we find that the ratio of the miles of concrete road built during the year to the miles of all other types of hard-surface road construction, is almost twice the ratio that existed in the total construction recorded at the beginning of the year. In other words, the concrete road is gaining on all of its combined competitors at a ratio of almost two to one.

Here we find the popularity of concrete comes not from economy of first cost, but from satisfactory service and low maintenance cost. These are characteristics that promise to make it the king of road materials. It is the prediction of those of vision in the industry there ere long the base of all main-line railroads will be of this magic material.

We have been taking thus far what might be termed an airplane survey of the peaks of the concrete industry. Every specialist may apply the same basis of analysis and measurement to his own particular activity. Growth and increased uses are found on all sides. To those who have been less analytical, it may seem that this has come about through natural laws and without system and well-directed effort. To us, who have had

some little part in building up the industry, it has been evident for some years that it is due to the earnest, conscientious and wise effort on the part of able men banded together in organization.

Outside of the large engineering societies three organizations have concentrated specifically on this subject. They are the Portland Cement Association, the American Society for Testing Materials, and our own Institute. Fortunately, there has been a very happy co-operation between these societies, which has naturally worked toward effective accomplishment. The Portland Cement Association, through its generous contribution to scientific investigation and research, its scientists of outstanding ability, and its wise and co-operative policies, has rendered assistance of inestimable value. The American Society for Testing Materials, although charged with the responsibility of all building materials, is ever alert and ready to contribute its effort to the upbuilding of concrete.

The Institute has endeavored to investigate every specialty of the industry and record each forward step. Some thirty-two committees are in constant operation. Each aims to record new information as it develops in the subject in which it has specific interest. This supplies to our members the latest standards and practices. To return to our original figure of speech, every new explorer is able through this method to use the charts of those who have gone before.

Thus far our eyes have been turned toward the receding horizon. Let us now look forward. To some it may seem that there are few yet unexplored Eldorados. However, careful analysis indicates that the future holds in store much that is unknown and of value.

We are considering in this convention the factor of time in the making of concrete. The developments from this research and investigation may in themselves contribute richly to the future of our industry.

When we observe the accomplishments of our European friends in the designing and building of a bridge having three 600-ft. spans, we recognize that we are only beginners in the school of concrete design and construction.

The value of the water ratio is receiving more and more marked recognition but as yet we have not touched on the sequence and methods of mixing, which in the opinion of some, will teach us much of economic and engineering value. And so we might catalog a score of unplowed fields—rich and susceptible of great additional cultivation.

The success of this organization has been due to the co-operative efforts of a large number of its members. No one man, no executive committee, no secretary, no committee alone, nor any series of committees, can carry forward our program. Each member has a part to do, whether he is on a committee or not. Every committee is endeavoring to keep abreast of the subject in which it is specifically interested. When a member finds facts that will contribute, it is not only his pleasure but his duty to report them, thereby making himself a scout in the forefront of our advance.

We are met here to examine and discuss the gleanings of this vast army of intellectual harvesters. It is our purpose to afford these conven-

tions an opportunity to present and discuss papers having to do with the outstanding accomplishments of the year. To our minds it seems better to direct our attention to these matters rather than to spend much time in the presentation and discussion of formal committee reports. These are all necessarily matters of record. They stand as the framework on which our structure exists, and are consequently of great importance.

In all that has been said there is cause for satisfaction: and there is also danger of complacency on our part. Philosophers have ever pointed to the perils of prosperity. They lurk in the various seas through which we are now passing. Vigilance and wise piloting must be our watchword. Each year must see a better product and a cheaper product, otherwise we are courting danger.

The manufacturer of cement must improve the quality of his output so that a better product may be forthcoming which will result in a saving in cost and time when measured in the finished structure. The fabricator of materials must develop methods and apparatus to accomplish the same result.

To those who know the industry, it is evident that the time and opportunity is right and our advance is certain if we are not thwarted by selfishness and short-sighted policies.

This, our Institute, is one of the armies that has the privilege of carrying the flag forward. Let us advance steadily with wise counsel and never-failing courage.

Notes on Concrete—Wacker Drive, Chicago.

By Arthur R. Lord.*

Synopsis.

The present is an era of specialization in concrete proportioning, in which the old standards are rapidly being discarded and new paths to new and better standards sought in many directions. Many new paths have been reported in the *Proceedings* of the American Concrete Institute among which those of Abrams, Young and McMillan are notable. This paper describes the path followed in proportioning and handling the concrete on an important public work in the city of Chicago, in which some partially appreciated truths have appeared to have much greater bearing on this matter than is generally realized.

Contrary to the opinions set forth by many past workmen in this field and now proposed for adoption as standards, this work strongly suggests that specification based on water-cement ratio alone will not be found adequate either to safeguard fully outdoor concrete structures or to produce optimum results. The problem of proportioning concrete is and must remain far more complex than the advocates of this standard lead us to expect.

Choice of Materials.—The influence of the varying qualities of the many brands of portland cement, all passing the present standard tests for cement, is far reaching on the strength and uniformity of the concrete. The influence of the stone of which aggregates are composed and of the gradations of the particles as produced from different sources and by different methods likewise is far reaching. A preliminary investigation of the properties of the available materials when combined in different manners is an essential step which must not be neglected if the best results are to be secured. In such investigations not one curve expressing the water-cement-strength relation will be found, but a whole family of related curves. From among them, that curve should be used which assures the desired strength, density and impermeability at the least cost. The discovery of the desired curve is relatively simple and inexpensive on work of considerable magnitude, and generally is productive of such economies as to make it practical even on relatively small structures.

* President, Lord & Holinger, civil and architectural engineers, Chicago. Consulting engineer to the Board of Local Improvements, city of Chicago, on the design and construction of the South Water St. Improvement (Wacker Drive).

Importance of Temperature.—The importance of temperature on the curing of concrete has been generally disregarded. The current practice of specifying a curing temperature of 50 deg. F. for three days, and of assuming that the concrete will ultimately gain its full designed strength from this cold start, appears to be unsupported by our data. It appears rather that curing temperature is a direct variable affecting concrete strengths at all ranges and comparable in its effect with the water-cement ratio itself. Temperature effects may nullify the savings indicated by water-cement considerations in the cooler months and on the other hand may greatly increase such savings in the summer period. Adequate research to discover the effect of temperature applied in varying degrees and at varying periods in the curing of concrete is needed to clarify the situation. It is assumed in this discussion that temperature is always applied simultaneous with moisture when used as a curing agent for concrete. Both the Abrams type of water-cement-strength equation and the Slater equation expressing the relationship between 7 and 28-day strengths are applicable in their present form only under constant temperature conditions. Both equations must be expanded to include proper temperature factors in order to be general.

Need of Controlled Consistency.—On outdoor concrete structures, and especially when such structures are heavily reinforced and of considerable vertical depth, the consistency must be controlled within far narrower limits than is now generally done or even advocated in present proposed standards. While a predetermined strength may be obtained with concrete showing a wide variation in consistency, every other needed quality of outdoor, heavily-reinforced concrete will be impaired by the use of sloppy concrete, even though the water-cement ratio is maintained. To secure dense concrete of uniform quality throughout its mass the consistency must be limited to the narrow range between that stiffness at which concrete begins to crumble and that wetness at which concrete begins to segregate. Such consistencies give a concrete that flows readily without separation. Places in which it is difficult or exceptionally expensive to place concrete of this consistency are almost invariably places where the loss of density, the increase in permeability and shrinkage and the segregation resulting from a sloppy consistency will prove serious dangers to the life and strength of such a structure. The use of sloppy concrete of whatever water-cement ratio will seriously affect design considerations such as bond and compressive stress at various parts of the structure.

To work within the range of consistency stated above it is necessary to control the grading of the aggregates and the water content of the batches more carefully than is commonly done. Aggregate producers now place unnecessary obstacles in the way of good concrete by their careless screening and lack of orientation to the well-established facts of concrete proportioning. This difficulty can be overcome at present by persistent and continual effort. Inaccurate measuring and poorly devised mixing and conveying machinery must be ruled out, and the key men in concrete mak-

ing, placing and inspection must be adapted to their work if uniformly high-quality concrete is to be placed in the finished work.

Precise Control Essential.—The problem of delivering concrete of a specified strength (or that of specifying concrete by strength) is tied up with precise control of all materials, of all manufacturing, transporting, placing and curing operations and of the methods of measuring the strength of the concrete in the structure. It involves a degree of technical control and precision in concrete manufacturing far greater than con-

FIG. 1.—WACKER DRIVE UNDER CONSTRUCTION IN 1926.

Aerial view showing the relation of the Drive to the Chicago "Loop" and to the North and West sides of the city. © *Chicago Aerial Survey Co.*

tractors ordinarily bring to construction work or are prepared to undertake generally at present. The necessary accuracy in determining the strength actually present in the concrete is not the least difficulty. Use of core boxes, in which concrete cylinders are embedded in boxes filled with concrete, and placed so as to have the same curing conditions as the slab itself, seem to offer the best measure of strength in the actual concrete aside from cutting into the structure. This latter is generally not permissible in a reinforced-concrete structure.

The greatest obstacle to the production of high-quality concrete is complacent satisfaction with low standards. In this paper is described an attempt to secure thoroughness and scientific accuracy in every detail of

a large construction program. The result was, at first, considerable mental disturbance and unhappiness for a lot of people and, finally, the creation in these same people of a marked sense of pride and satisfaction in their ability to turn out a superior product. The development of a sense of pride is as essential to the success of such an enterprise as is the command of the necessary technical skill and financial resources. Special privilege arising from political pull, is fatal to the necessary *esprit de corps.*

As a result of careful preliminary tests, of rigid control of materials, of proper machinery for measuring, mixing and transporting of concrete, of the careful selection of the key men operating this machinery, of loyal and intelligent inspection, of careful control of the consistency, placing and curing of the concrete and of placing of the reinforcing steel, of technical direction fully informed of what had been accomplished by others in this line, and of freedom from political interference assured by the deep interest of the administration in the welfare of this undertaking, it is believed that the concrete work on Wacker Drive was exceptionally well performed and will prove to be both strong and enduring.

INTRODUCTORY.

The general nature of the South Water St. Improvement, involving the demolition of the row of buildings abutting on the river and the use of this property together with Old South Water St. for a new double-deck boulevard, is familiar to members of the Institute through the writings * of T. Arthur Evans, engineering advisor to the Board of Local Improvements.

I became associated in this work upon Mr. Evans' request and recommendation, at a time when the South Water St. ordinance, containing preliminary plans and specifications for the work, had been completed and passed by the city council and was before the courts for final action. My connection with the design was therefore very slight, consisting largely of consultations with Mr. Evans and his designing assistants while the final construction drawings were being prepared from the ordinance drawings. I may state, however, that the design was made with great care and thoroughness, employing the slope-deflection method of analysis and taking into account probable variations in load distribution as well as expansion and contraction movements due to moisture and heat. Stresses under direct load were limited to those specified for flat-slab construction in Chicago, but for the final all-inclusive stresses values up to twice these limitations were permitted and used in some cases. These very highest stresses occur only in corner columns or other occasional locations and are rarely determined with accuracy in most similar designs. But the presence of high stresses locally and the repeated experience in the past of increasing wheel loads made it highly desirable to produce on this work a concrete of far more than usual strength but at the usual price—in other words, scientific

* See p. 259, A. C. I. *Proceedings*, 1926: also *Journal* Western Society of Engineers, March, 1925.

concrete. I was brought into the picture largely for this purpose—to advise the engineers of the board on all matters relating to concrete materials and mixtures and to the manufacture, placing and curing of 120,000 cu. yd. of concrete in the field. This paper will deal largely with that phase of the enterprise. The opinions and conclusions herein set forth are mine and no responsibility for them attaches to the Board of Local Improvements nor to its technical staff.

I was assured that I would be free to apply scientific principles in the work without restrictions except as imposed by laws relating to special assessments work and by the professional judgment of my associates. While the results as a whole have attracted favorable comment, I feel personally that the most notable aspect of this work has been the failure of every attempt to influence the concrete making by political means. Without such freedom from political abuse scientific concrete would be impossible of achievement on public work.

PRELIMINARY CONSIDERATIONS.

An outdoor viaduct extending for a mile along the water's edge and built of heavily reinforced concrete offers one of the most interesting and difficult problems that can come to a concrete specialist. A concrete building frame, encased in brick and terra cotta and comfortably heated in winter, may endure even though it is made of relatively porous and weak concrete and even though its reinforcing bars may be barely hidden in the surface. But on Wacker Drive we faced the facts that every bad batch of concrete and every misplaced bar would some day rise up to testify to our lack of thoroughness. Therefore, at the beginning of our work we formulated a number of "points"—our *articles of faith* as it were—with which we must reckon on this work, and these are set down for comparison with our later experience, which leaves them largely unchanged, although new ones are added.

1. *All* the concrete in an outdoor structure should be strong, dense and impervious, while every part of every reinforcing bar should be protected by at least 1 in. and in some cases by 1½ to 2 in. of good concrete or mortar.

2. While good concrete may be made from a wide variety of materials, a careful study of the concrete materials available for this work will be necessary to get the best results without increase in cost.

3. The source of each material must be such as to insure an adequate supply of material of constant qualities so that a mix once determined can be continued through each stage of the work. Unless the contents of each batch can be successfully standardized and maintained uniform it will not be possible to keep to that degree of stiffness which is most desirable.

4. Sufficient time should be reserved before field operations to make preliminary tests of *concrete* from available materials. These tests need not be elaborate or expensive but they should be thoroughly well-made.

5. While the water-cement ratio is of primary importance, as indicating strength of workable mixtures, this relation is not the same for all cements and all aggregates nor for all gradations of aggregate from the same source. It must be determined in advance for the particular materials to be used.

6. For outdoor concrete, workability must be limited to a very narrow range, between that stiffness where concrete *begins to crumble* and that wetness where concrete *begins to segregate*. This range, measured as slump, will vary greatly with different aggregates and should be determined in the preliminary tests.

7. The equipment is hard to change when once installed and the best kinds should be selected. Some mixers refuse to give up a batch of very desirable stiffness. Some conveying devices absolutely insure segregation, except under extraordinary circumstances. Measuring devices must be such as to insure repetitions for all batches.

8. An adequate mixing time should be maintained rigidly. As long mixing requirements naturally involve large-size mixers, and as large-size mixers in turn require longer mixing time, the usual specification of one minute is too small rather than too large.

9. The workmen largely influence the concrete. Men who by temperament or habits are opposed to the manufacture of concrete in accordance with an exact, unvarying recipe, must be eliminated from key positions.

10. Having spent time and money in producing a uniform concrete at the discharge of the mixer, segregation during transportation and placing must be prevented.

11. Freshly placed concrete must be protected from drying out. Curing conditions involving adequate heat and moisture supplied simultaneously and continuously for at least the first seven days must be insured, if strong and impermeable concrete is to obtained.

12. Inspection must be adequate to insure that:

(a) Every batch that goes into the forms is properly proportioned and mixed.
(b) All spaces within the forms are filled solid full of concrete.
(c) Every bar is away from the surface, as shown on plans.

13. Inspectors should be inspected and assisted. This meant in our case that either Mr. Evans or I should be on the work at all times. With a 24-hour daily schedule we were not able to live up to this fully.

14. Great consideration is desirable before anything is added to the indispensable contents of concrete. Of all the 200-odd admixtures offered for incorporation in concrete, political interference is the worst.

Men believe a thing, says Spinosa, when they behave as though it were true. Our practical problem on Wacker Drive was to make everybody connected with the work *behave* as though these fourteen points were true.

EARLY EXPERIENCES.

Our first construction specifications required that the materials to be used for concrete were to be selected by the contractor from a source offering a supply of uniform qualities and adequate in amount for the work under the contract. City engineers gathered their own samples from available sources and a period of five weeks was reserved between the selection of the material and the use of it in concrete on the job. This arrangement was believed to be fair to the contractor in that it permitted him to buy in his best market, material meeting our specifications for cement, sand and stone or gravel, while it left us free to use our utmost ingenuity in combining the materials selected so as to produce the strongest and densest concrete possible with them. This period was later reduced to three weeks, as 28-day tests are not necessary as a part of such preliminary tests. Proportions were stated as 1:5, 1:6, etc., with the provision that the volume of sand might be as low as one-half of the coarse aggregate or as high as 65 per cent, the exact proportions between these limits being left to the engineer to determine. The water-cement ratio was limited to not over 7½ gal. of water per sack of cement, (water in the aggregates being included in the measurement) for the 1:6 mix and corresponding values for other mixes. Our experience in applying this kind of specification to public work was sufficiently interesting to warrant a brief recital here.

Procuring Test Materials.—First we proceeded to look for samples of the cement which was to be held for this work while we made the standard tests as set forth in the specifications. The cement producer who had the contract suggested that we proceed to gather our samples from the mill as their product was being sacked and shipped to various points in the Central West. We could see how this would assist some enterprise in Indiana or Iowa, if they could obtain our test results, but hardly how it would assist us or assure proper cement on Wacker Drive. An appeal to political pull was in order and promptly forthcoming. Mayor Dever or someone close to him will have to write the story of the ensuing week—all I know is that at the end of it 50,000 bbl. of cement went into storage for our use while we took our samples and made our 7 and 28-day tests. I have reason to believe that our cement was no worse on account of that skirmish. As the cement traveled along on the belt to the bins set aside for our use the indentation made by the city samples was promptly neighbored by two additional indentations. We had no occasion to reject any cement on the entire work. Each truck load of cement brought with it the shipping tag of our mill inspector with the serial numbers of the test samples of that bin.

Our early concrete also required sand and crushed rock. We asked where the stockpiles (it was December and crushers were not operating) set aside for our use were to be found. Some little bird had hinted to this first contractor that it would do no harm to purchase his aggregates from a certain firm, even though materials from that source might cost a bit more per yard; the advice sounded reasonable and was followed. An

ample stockpile was duly dedicated to our exclusive use, samples were taken, tests of fineness, voids, weight, specific gravity, absorption, bulking, decantation, impurities and density of mixed aggregates were made and we had started making 6 x 12-in. cylinders and trial batches, when it occurred to us to have a look at the stockpile. It was gone! We ruefully surveyed our elaborate calculations à la Abrams and à la Talbot and concluded that we must find some quicker method of proportioning concrete if we hoped to keep up with our material dealers. Our date for starting concreting was a week nearer but we were exactly where we started. History repeats itself—sometimes with bewildering rapidity. Other stockpiles were solemnly dedicated, other tests started but again those substantial hills took wings and vanished. I suspect that certain politicians were doing their best to show up the absurdity of our specifications and professional methods. We did manage to get 7-day tests and started the foundation work. That concrete went into great open caisson wells from 4 to 6 ft. in diameter where very stiff concrete was an advantage in every way and strengths were very satisfactory, even though we had to handle and "proportion" sand and stone gathered from nearly every yard in Chicago in the course of two months. We gained an encyclopedic knowledge of Chicago concrete materials.

While we could and, by severe application, did make high-strength foundation concrete under these conditions, we realized that in the heavily-reinforced deck slab we were lost under such conditions and that the issue must be joined on foundation concrete. Conferences in Mr. Sloan's office produced the fairest sort of promises but failed to pin down further appointed stockpiles. Then this material firm went off the job; and stayed off for three days. While that discouraged us at the time, we realize now that it was best to give this company every chance to make good. When they promptly resumed delivery on precisely the same terms, they went off again; and this time they came back no more on that section. On a later section the same company was permitted to try again, and it delivered entirely satisfactory material over a period of several months, complying with our requirements cheerfully.

During the three days above mentioned we located a supply of aggregates of very good characteristics and started tests. When the political outfit was finally routed, we went to this source which carried us along nicely for two months in the winter season, when demand other than ours was slack. They had a long haul, however, and asked to be relieved in order to serve their close-in customers at the beginning of the busy construction season. We reluctantly let them go. With ample notice we searched out a new source of even better material, giving us materially higher strengths than our previous aggregates; and by far the larger part of the Wacker Drive concrete, including almost all of the superstructure, was made with this crushed stone. We were now free on much of the work to accomplish our end of making good concrete and to devote the bulk of our time to the inspection and control of the field operations.

We were not yet entirely out of the political woods, however. As the work progressed each new letting brought another contractor on the drive and each new contractor had his favorite material yard. On one section the crushed stone varied considerably in grading and quality. We had to reject many truckloads containing soft, porous limestone, some pieces of which could be readily broken with the fingers. As on the first section we put up with a good deal of changing about on the foundation work where we nevertheless secured excellent strengths, the more readily as this contractor used approximately the same cement in his 1:8 mixes as was required in 1:6 mixes. But with the superstructure approaching we had finally to clamp down the lid on varying aggregates and to demand a fixed source and constant grading of stone for our tests for superstructure concrete proportions.

We made our tests and the day came to place the first deck slab concrete—a 29-hour continuous run, starting at 7 a. m. In the first two hours of that period we received four distinct kinds of crushed stone, only one of which, naturally, represented the material on which our tests were based. The others were not suitable and were rejected. One kind contained large amounts of the soft stone mentioned above, another contained between 17 and 20 per cent by weight of crusher dust, while the fourth was made up almost exclusively of material between the ¾-in. and 3-in. screens! The rejected trucks lined up in the next block and made an imposing array while awaiting our capitulation. Naturally the job limped and the contractor became restive with so many men and so little concrete. That concrete was well spaded! The stone producer was hastily summoned as the job subsided into rest for lack of stone, and proceeded to express his views of scientific concrete and city "experts." He informed me that Professor Abrams would tell me how good his materials were and I graciously agreed to let my old friend come and rescind most of his published opinions if he cared to. He was not called. The producer then informed me that he was through, that not only stone, but sand and cement as well, would stop. I remained perverse although I felt, and told him so, that his sand was quite the best we had on the Drive, with four contractors working at the time. For an hour we marked time, after which the contractor secured his stone from the source last mentioned in the preceding episode. We continued to receive sand and cement from the producer whose stone was rejected. This man further stationed four men on the job to assist us in inspecting the stone from the rival concern—all at no cost to the city. Later on all the sand on the improvement was handled by this producer and we had but little further trouble with our concrete materials.

Stone producers have been and still are slow to make their product lend itself to high quality concrete. They make it unnecessarily difficult for their customers to succeed. We had a further difficulty that illustrates this backwardness. Our specifications permit 1½-in. stone, graded down to sand sizes, in caissons. Contractors have found no dealer in Chicago prepared to furnish this material. It can be had only by loading a truck

successively from three different bins and trusting to luck that somehow all three sizes will get into each batch of concrete in the right proportions. As in other cases of trusting to luck one may be sure of almost never winning. The other alternative is to use coarse aggregate ranging from 1½ in. to ¾ in. in size with a gap in grading all the way down to the sand. Concrete made from such a combination has the mobility of candied popcorn. It presents, in the forms, large stones coated with cement, with holes between, and with most of the mortar running all over. Satisfactory strengths can be obtained but segregation is inevitable and facing out of the question. Until producers produce a properly graded stone in the larger sizes Chicago specification writers are advised to stick to "concrete size" stone even for massive work. No contractor will install two or three coarse aggregate hoppers and batchers for the proper handling and measuring of such material as now produced, so far as we have found.

PRELIMINARY TESTS.

Cement was sampled at the mill as the storage bins reserved for this work were filled a month or two in advance of the job requirements. These samples were halved and a half of each tested in the standard way by Dr. Eaton of the Board of Local Improvements' laboratory and by Mr. McArdle of the laboratory of the division of engineering, Bureau of Public Works. We did not find here (and I have never found in my tests) the highly desirable uniformity in strength that commercial testing laboratories so commonly report. All our samples passed with flying colors but a very wide range in strengths was found—fully as great as that commonly shown in concrete cylinder tests; 28-day standard mortar briquette tests (average of 3 specimens) showed a range of from 337 to 507 lb. per sq. in. in samples from a single bin. Portland cement is a material produced by the ton by an elaborate and expensive process and sold at an extremely low price. A high degree of uniformity is not reasonably to be expected, nor should it be expected in concrete made from cement. The economic balance at present leads to a concrete whose weakest portions may be made to yield a satisfactory strength and density and that is all that we may expect or require on practical work.

To determine the usual run of materials, sand and coarse aggregate were sampled at the storage bins of the producers from time to time and checked by other samples collected on the job. Great care was taken to get representative samples. Generally from 500 to 1,000 lb. were taken and smaller samples made by successive quarterings. For preliminary test cylinders the aggregates were dried and the coarse aggregate separated into three sizes (0-⅜ in., ⅜ in.-¾ in., and ¾ in. up) and stored in separate covered boxes after weighing. The smallest size was further sampled and screened to determine the fineness modulus. At first we made every test on the standard calendar and a few of our own besides, but, as related above, this became burdensome with disappearing stockpiles. We finally reduced our efforts to a far simpler but adequate program which may be described as follows:

Making Test Pieces.—Take adequate pains to secure representative samples from the source of supply (about 1,000 lb. coarse aggregate and 500 lb. sand). Dry these samples and screen the coarse aggregate into three sizes as mentioned above, keeping record of weights. From the 0-⅜ in. size take a 1,000-gram sample (by quartering) and determine the fineness modulus of the coarse aggregate. In like manner determine the fineness modulus of the sand. Fineness modulus is easily obtained and frequently inquired for, although not particularly useful in the test method suggested here. You need to know the dust content in the stone, and by that time you have determined fineness modulus. Store each size of aggregate separately, keeping it always dry. Make decantation and colorimetric tests of sand and set aside for later observation and record. Determine the weight, loose and dry rodded, of each aggregate separately. For this purpose you must recombine the coarse aggregate in the observed proportions by weight and remix thoroughly. Now obtain the unit weight of various mixtures of fine and coarse aggregate (35 and 65, 40 and 60, 45 and 55, 50-50), drawing a curve through the platted results to determine the proportions giving maximum unit weight. We have found our best field mixture generally contains slightly less sand than this maximum-unit-weight combination and these data therefore reduce the range necessary to be covered by test cylinders later on. Determine the percentage of voids in each aggregate. If fine aggregate is to be used without inundation, make bulking measurements, starting with dry sand and gradually increasing the moisture up to 15 per cent. This gives the field correction to avoid variation in consistency of concrete due to change of sand volume with moisture content.

Study the results and proceed to make cylinders. After a little experience three sets of cylinders having three proportions of sand to stone will cover the field but at first five sets are desirable. Make four or five duplicate series, completing each series before repeating with the next. Using three water-cement ratios for each of three mixes gives 45 cylinders to make which is about a day-and-a-half's work. The whole testing operation thus far has required 3 to 4 days with one man experienced in such work and one helper. The character of the work for which concrete is needed will determine the consistency to be sought in the cylinder batches. In the first set of cylinders water can be added and the concrete remixed until the desired consistency is obtained, but later batches should not be handled more than once.

For each cylinder make up a batch totaling 24 lb. of cement, sand and coarse aggregate (weighed out separately and all weights recorded to minimize the chance of errors). Use a mixing pan with upturned edges, that is tight and that can be easily and completely scraped clean. Measure the water with equal care and add it all at once after the dry materials have been thoroughly mixed. Mix again thoroughly and make a standard slump test—only lift the cone at once instead of waiting 3 min. Observe the mixing carefully. Does the mass work smoothly or is it harsh? Does

it tend to segregate—either water or mortar running out or stones showing bare? Discard mixes not convincingly workable. If you plan to use lime, celite or other admixture in the field, use it in these tests also. When satisfied as to workability make the cylinder, being careful to *waste no concrete.* Scrape up every bit of concrete not in the mold and measure this remainder—the dust pan of the screen set makes a good measuring pan for this. The contents of the cylinder plus this residue gives you the total volume for the batch—a most important item when designing outdoor concrete where density is required. Repeat until all cylinders are made. Mix up some neat cement and water into a medium thick paste and come back after 5 or 6 hours and cap the cylinders when the first shrinkage will have largely occurred. Note the amount of this shrinkage as it, too, is important practically. We prefer metal molds, with plate glass or planed metal base and cap plates and we put a piece of paraffin paper between the cylinder and the end plates. Strip the cylinders in the morning, weigh each one carefully and store in damp sand at a room temperature of 65 to 70 deg. F. Temperature will affect your strengths as definitely and as greatly as the water-cement relation. Test two sets of cylinders at 7 days, damp as they come from the sand, and hold two for 28-day tests. If the 7-day cylinders vary widely test a few more cylinders at this age to get a better indication. From Slater's equation $S_{28} = S_7 + 30\sqrt{S_7}$ you can compute closely the 28-day strength from the 7-day tests.

Shrinkage Observations.—While waiting for the 7-day test or, better still, after these tests if time permits, try out some large size batches in the field and observe workability and shrinkage. A form with several bars rigidly supported near the top is a good arrangement in which to observe shrinkage—look for the open space under the bar as the concrete is chipped away. Your bond depends on keeping this very small. Cut out some of this concrete, saturate it thoroughly, determine its volume by immersion in water, after thoroughly saturating, and then dry to a constant weight. Compare the unit weight for this concrete with the weight found in the cylinders. In making such field specimens be careful not to overdo the spading—try to reproduce working conditions in every respect. If a concrete saw or a coring machine is available, sawn prisms or cored cylinders, twice as long as their least diameter, will give a check on the laboratory cylinders—provided curing temperatures have been the same.

By substituting volume for weight measurements in cylinder making this system may be made even more direct, but weights are commonly used in the laboratory and often in the field. The separation of the coarse aggregate into three sizes is important, if small batches are to give reliable indications of workability.

On small buildings, where all conditions hold out promise of a legitimate saving in cement and where density is not so important, I have used a straight 1: 6 mix in starting the foundations and taken cylinders of this

for 7-day tests. From these tests the Abrams curve for the materials used may be established like the dash line in Fig. 2 and from this curve a *safe* estimate may be made as to the possible reduction permissible in the cement and this in turn checked up by further tests. Thus, by steps, the mix may be brought down to the economical basis without danger of producing either a weak concrete or an unworkable mix. In such experiments the shrinkage of the cylinders should be recorded when they are capped (about 5 to 6 hours after making). It is very simple, too, to vary the proportions of sand and stone in a few batches and also the slump and

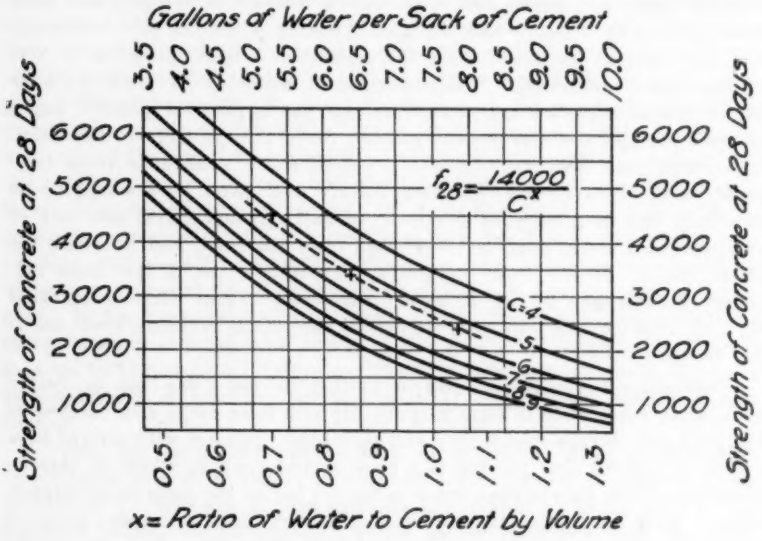

FIG. 2.—ABRAMS CURVE DETERMINED FROM JOB CYLINDERS.

Chart saves logarithmic calculations in the field. Job cylinder strengths plotted against water content determine the curve for the particular materials used. The dash curve may then be followed to desired strength. Proper allowances must be made for changes due to temperature.

so obtain cylinders for comparison. In all cases the cylinders should be weighed, before being stored in damp sand. The method of casting four or more cylinders in paper molds surrounded by concrete in a box is also readily applied in crude but very helpful field experiments of this sort. Such abbreviated methods should not be used for outdoor concrete where density and impermeability are fully as important as strength.

WORKABILITY AND STRENGTH.

Except for certain limitations in the proportions of coarse and fine aggregate, water-cement ratio specifications require only a definite water-cement ratio (as a control of the strength) and a workable mix. A work-

able mix is defined as one that can be "puddled readily into the corners and angles of the form and around the reinforcement without excessive spading and without segregation. . . ." As a further control of the workability, a maximum slump of 7 to 9 in. was specified for the different classes of work on the Portland Cement Association Building.* I disagree with this method in the matter of the maximum slumps permitted. I believe that more emphasis should be placed on the importance of plastic consistencies, as limited in this paper. Slumps of 8 or 9 in. as were used on this building, with the ordinary aggregate combinations, cannot be expected to result in durable concrete for outdoor exposure, regardless of the water-cement ratio maintained at the mixer. Further, they cannot be used without segregation.

With the same water-cement ratio a medium plastic or stiff concrete is superior to a wet or sloppy concrete because it is denser, heavier and less permeable. There is less diffusion of its particles, produced by water. It is more uniform and segregates far less in transportation and placing. It shrinks or settles very much less and grips the reinforcing steel more strongly. It is less subject to damage by freezing and thawing.

Honeycombing.—Medium stiff concrete presents far less danger of deep or hidden honeycombing, such as would affect the strength of a beam or slab. Honeycombing has come to be considered almost entirely as a blemish in surface finish. It is far more than that. Dangerous honeycombing is caused by the mortar separating from the stone in wet concrete and leaving what is really a weakly-cemented stone or gravel pocket. This sort may or may not appear on the surface. On one job, for which I was called in to prescribe, a careless plumber had cut a hole through a wall and uncovered a stone pocket which crumbled as soon as the nice mortar finish, made by surface spading, had been broken through. Investigation showed that this entire foundation job, both walls and footings, was honeycombed under its false face. Drills struck water practically anywhere—even in the middle of footings 20 ft. square and 6 ft. deep—at depths of from 1 to 3 ft. The entire mass was traversed by water passages through connecting volumes of hidden honeycombing leading to the ground water outside. Much of this concrete had to be removed—the strength of the structure was gone on account of the wet consistency and its attendant train of evils. This sort of trouble is inconceivable when concrete of the consistency advocated in this paper is used. With concrete of the consistency here advocated one occasionally finds a *surface roughness* caused by the failure of the mortar to flow out freely against the forms and which a slight amount of surface spading would have prevented. But surface roughness of this sort does not materially affect the strength and life of the structure, while the type of segregation caused by sloppy concrete is the usual cause of rusting reinforcement and structural weakness when these occur. If the pockets of honeycombing occur at regions of high stress the results are structurally very serious and may cause failure.

* *Proceedings*, American Concrete Institute, 1926, p. 122.

I can see no good argument in favor of mixing concrete thoroughly and expensively only to have it un-mix rapidly the second it leaves the mixer. In fact, with most mixers discharging into a wheelbarrow or cart direct and to a less degree when discharging the entire batch into carts through a floor hopper, with sloppy concrete the various cart-loads from the same batch will vary greatly in quality. On any length of haul the stone in the wet batch packs on the bottom of the cart and the mortar runs off first when the cart is dumped. On Wacker Drive we were unable to pull out formwork tie-rods after 16 to 24 hours, which will pull out readily even after a week when sloppy concrete of the same water-cement ratio is used. This ease in pulling ties may seem an advantage to the contractor but it bespeaks a serious defect in the concrete, which has settled away from the steel so as to leave but a portion of its surface in contact with the concrete and thereby has largely reduced the bond value, especially with plain bars.

Consistency.—Another fundamental weakness of wet concrete even when a low water-cement ratio is maintained, is found in the variation through the mass as finally placed. With a 9-in. slump, or even with a 7-in. slump with our Wacker Drive materials, excess water and mortar would gather on the surface, sometimes to a very considerable depth. In such concrete the water-cement ratio is no longer constant through the mass. At the bottom it is much less than the specified ratio; at the top very much more. In deep sections this becomes especially serious. This means weak joints and laitance, and while the extra cement helps some, the condition bespeaks uneconomical use of cement and labor. It is the same old fault that has brought decay to horizontal construction joints and caused the tops of walls and columns and abutments to shear off or weather away. This top concrete is poor stuff in a fire, also. It is poor stuff to rely on to furnish the bond for negative reinforcing steel in continuous structures and to take compression at midspan. It affords a most insecure base for a concrete floor topping to adhere to.

On the practical side I doubt if the contractors would have gained any material advantage had we permitted them to use an 8 to 9-in. slump instead of a 3 to 5-in. slump. I am certain they would have had a lot of patching to do. After they once were broken in to handling stiff concrete and the men felt that their bosses were not fighting it, we heard few regrets over the "good old days." The contractor who co-operated from the start in handling our stiffer consistencies, kept cutting his prices on every successive letting, which would indicate a rather passive resistance to our dry concrete. I feel that concrete of 4 to 5-in. slump is as easy to handle as any other consistency and that on ordinary work, where the reinforcing steel is not so heavy and so close meshed, such stiff concrete would compare favorably with any other in cost of placing, considering patching as a part of placing.

Our experience on this work covering a period of nearly two years, working 24 hours a day in our superstructure concrete placing, continuing

through every sort of weather, leads us to advocate consistencies such as we used, as being as good in every way and better in almost every way than concrete with the same water-cement ratio and more water. We started in 1925 with an average slump of about 5 in. Gradually this was reduced to an average of 4 in. and on some placings it averaged as low as 3 in. in 1926. I have never had a job, with the single exception of the concrete ships, where steel was more congested or placing generally more difficult than on this one. If I were doing it over again I would certainly stick to this kind of concrete on every count. I feel that there is a great advantage in getting a high strength with a 1:6 mix and a stiff consistency rather than with a 1:4 or 1:5 mix and a sloppier batch. The extra cement would certainly add extra shrinkage. I am quite ready to agree that concrete must not be *too dry*. The limits of proper consistency seem to me to be quite sharply defined. As the mixing water is reduced the concrete flows less and less readily until finally it *crumbles* rather than flows. Unless concrete is to be consolidated by tamping or hammering a crumbling consistency cannot be placed satisfactorily. Our concrete (1:6 mix) crumbled at about a 1-in. slump. By working in the range of slumps between 2 and 4 in., we kept our concrete mobile and self-compacting while securing exceptional strength. Now consider water (or water and cement) added to our usual batch. It rapidly becomes wetter until at a slump of 6 in. or slightly less the mortar visibly runs away from the stone. In a 17 to 24-in. slab the mortar rises to the top, the stone settles to the bottom and after a short while free water covers the top. Segregation has begun and the slab is no longer uniform. In designing such a slab one ought to use much lower bond and compressive unit stresses at the upper surface than at the lower surface to have a balanced design. Crumbling and segregation mark the two easily observed limits of concrete consistency. The corresponding slump values will vary with the particular materials used. The slump limits should be determined from the proportioning tests for the materials to be used on the work and the final specification should leave a margin at each extreme to accommodate reasonable and unavoidable variations in the materials. Our working limits were 2 to 5-in. slump; and the less 5's we had the better we liked it.

One final observation on consistency. I have a greater respect for slump after this job than ever I have had before. With good control of aggregates the slump, the strength and the water-cement ratio move together surprisingly well, and throughout the job the slump test proved a reliable guide in controlling the work. When weather adds or subtracts an increment of water between the mixer and the forms, the slump measurement is the only measure of the final water ratio. At the least, the slump cone is a great arbiter of differences of opinion and a safeguard against futile debate. Until we find some better measurement of consistency it is well to specify slump tests, leaving the range to be determined by experiment. We used a flow table in the laboratory but did not find it suitable for field control work.

MATERIALS USED.

Materials Specifications.—Our specifications for materials followed closely the standards set up by the 1921 Joint Committee and by the American Society for Testing Materials. Portland cement was required to meet the A. S. T. M. specification C9-21 with the added requirements that (1) the 28-day strength must exceed the 7-day strength by at least 20 per cent and (2) the record of the particular brand of cement supplied must have been satisfactory under conditions similar to those obtaining on this improvement. The last provision was intended to eliminate cements which set with extreme slowness in cold weather or have other known practical objections. Most of the cement used on this work was Universal brand, trucked in from the mill at Buffington, Ind.

Fine aggregate was limited to sand with the usual descriptive adjectives affixed as given in the Joint Committee report. All tests, including fineness, decantation and colorimetric, were required to be made in accordance with A. S. T. M. standards, C41-24, D136-22T and C40-22. Grading was specified in accordance with the following proportions by weight:

No. 4 sieve, not less than 85 per cent passing.

No. 50 sieve, not more than 30 per cent and not less than 10 per cent passing.

No. 100 sieve, not more than 2 per cent passing.

Decantation test, not more than 3 per cent removed.

Coarse aggregate was limited to crushed stone in most of the viaduct structure, on account of the more satisfactory behavior of stone-concrete under temperature changes. This difference in volumetric changes may be due to the marked difference in grading commonly found in the two kinds of aggregate. In Section 12 and for the structural steel incasement on Section 2, gravel was used, however. Grading was specified as indicated in the following table, tests being made in accordance with A. S. T. M. specification C41-24. This grading departs somewhat from the similar table in the Joint Committee report for the reason that we found no source of supply that would give us material of that grading.

Coarse Aggregate Designation	Percentage by weight passing through Standard sieves with square openings				
	3 In.	1½ In.	¾ In.	⅜ In.	No. 4
No. 1				Min. 95	Max. 5
No. 2			Min. 90	20-40	Max. 5
No. 3		Min. 95	20-40		Max. 5

No. 1 size was used for fireproofing and encasing structural steel. No. 2 was the usual concrete stone. No. 3 was used in caissons when it could be obtained.

A period of at least three weeks was specified for sampling and proportioning tests before any material selected could be used in the structure. The moisture content in the coarse aggregate was limited to not over 2½ per cent.

For sidewalk topping, granite screenings were specified, graded so that at least 95 per cent by weight would pass a No. 4 sieve, not more than 20 per cent would pass a 50 sieve and not over 5 per cent would pass an 80 sieve.

Reinforcing bars were specified as intermediate grade, new billet steel made by the open-hearth process and complying with Joint Committee specifications for this material. We had a few lively sessions with various gentlemen who undertook to furnish steel to this specification and whose stockpiles were badly mixed. Some companies were more interested in explaining the unexplainable than in living up to our specifications and these soon followed like-minded stone producers into the discard. From every dealer we received badly-bent steel. Common serious faults were oversize stirrups and bulges in spirals at splices, and these caused a lot of extra work. The bulk of the slab steel was received straight and bent by use of "hickies" after being placed.

Hydrated lime was specified (50 lb. per cu. yd. of concrete) except for massive concrete and use in cold weather, when extra cement was used.

Concrete mixes were specified as follows:

Designation of Mixture	Parts by Volume			Water, gallons per sack	Part of Structure
	Cement	Aggregate			
		Fine*	Coarse*		
1:7½	1	2.5	5	7.5	Cylindrical piers as specified.
1:6	1	2.0	4	7.0	Structural concrete work in general.
1:4½	1	1.5	3	6.5	Columns and column capitals of viaduct structure, when so shown on drawings.

* "In these two columns are stated the usual nominal proportions. The exact division of the total aggregate between fine and coarse aggregates will be determined by the Board of Local Improvements so as to produce the required strength and density. The volume of sand may be increased as much as 30 per cent, depending on the character of aggregate furnished for the work, but such increase in fine aggregate shall be balanced by an equal reduction in volume of coarse aggregate.

"1:6 concrete shall be used for any portion of the work for which the contract drawings and specifications do not give the mixture of concrete. In case of conflict, the richer mixture shall be used.

"The amount of water given in the table is inclusive of the moisture in the aggregate. The contractor may increase the water content of any mix at his own expense, provided that the water-cement ratio shall be maintained constant and provided other requirements of these specifications are met."

Measurements of sand were made in the inundator which gives dry-rodded volumes. Measurements of stone were made in the batcher which gives damp-loose volumes. On a dry-rodded basis the sand in our various mixes varied from 38 to 46 per cent of the total aggregate by volume and from 41 to 49 per cent of the total aggregate by weight.

The last paragraph quoted above from our specifications is the McMillan specification used on the P. C. A. building in Chicago, but the following

sections removed all danger of shrinking, porous concrete by limiting the maximum slump to 4 in. As used here the provision simply required extra cement as well as water if the materials selected by the contractor wil not produce a workable concrete (3 to 4 in. slump) with 1: 6 mix.

"Retempering," defined in our specifications as "remixing with added water after the concrete or mortar has partially set," was forbidden. Much confusion, among Chicago contractors at least, has resulted from the use during the 1926 A. C. I. convention of this old term "retempering" to mean the beneficial remixing of concrete *without* added water. The way such matters repeatedly come up in most unexpected quarters indicates the great influence the Institute exercises over non-members and imposes an obligation to be precise in our statements.

Grading of Materials.—The following table gives representative gradings for the run of material as used on this work. "HR" was the designation of the Lincoln St. quarry of the Chicago Union Lime Works Co. from which the stone came which gave highest concrete strengths. "IG" designates our best gravel from the Reid Pit in Elgin and "aS12" our best sand, from the Consumers Co. Pit at Beloit, Wisconsin. We always found some fine materials in our coarse aggregate although it was very clean stone.

	Per Cent by Weight Retained on Sieve Indicated									
Indentification	1½ In.	¾ In.	⅜ In.	No. 4	No. 8	No. 14	No. 28	No. 48	No. 100	Fineness Modulus
HR12		6.3	83.4	93.9	95.5	95.9	96.2	96.9	98.7	6.67
HR16		11.1	64.7	95.7	97.2	97.7	98.0	98.2	98.5	6.61
HR22		11.1	69.4	91.3	98.8	99.1	99.2	99.3	99.4	6.68
HR27		9.6	62.6	85.8	95.5	97.4	98.0	98.7	99.9	6.48
IG2		8.6	64.8	82.8	92.9	95.7	97.2	98.5	99.3	6.40
1G6		19.3	84.1	96.9	98.7	98.9	99.3	99.5	99.6	6.96
aS12				4.7	15.8	27.8	49.5	92.1	99.7	2.90
aS16				1.5	10.5	23.6	48.0	90.0	99.3	2.73
aS20				2.6	13.0	23.0	48.3	89.5	98.2	2.75

The above values are from routine tests. We also have many tests taken when appearances led us to feel that the material was beginning to depart from the range covered in our proportioning. On such tests we found extreme values of the fineness modulus running up to 7.25 and down to 6.20 for the coarse aggregate. Similarly, values running up to 3.0 and down to 2.5 were occasionally found for the sand. The weight of the stone (dry rodded) varied little from 99 lb. per cu. ft., while that of the sand varied from 111 to 114 lb. per cu. ft. Voids in the regular sand averaged 31.5 per cent and in the stone 41 per cent.

We objected strenuously to high percentages of dust and generally had no difficulty in getting clean stone. Inasmuch as there is a good market for limestone dust we had no lengthy arguments over enforcement of the usual 2 per cent limit. Dust, if present, in any considerable quantity, has the habit of gathering in pockets with unpleasant consequences when a pocket is encountered. Just as the necessary workability has been restored for the dusty stone, the pocket will be exhausted and the concrete wil suddenly turn sloppy.

PLANT AND EQUIPMENT.

No detailed description of the various plant arrangements used on Wacker Drive will be attempted, since my purpose is to discuss plant as it helps or hinders the making and placing of highest quality concrete. With five contractors manufacturing concrete on the various sections of Wacker Drive a variety of plant arrangement was naturally used. The yardages were large and a good plant was justified. The Mid-Continent Construction Co. had more of this work than any other and their plant is perhaps of the most interest. Mid-Continent had the first contract (Section 3) and was the first in Chicago to use the inundator. This device was specified for the work although it was not placed on the market and could not be used on our work until March, 1925. The first plant therefore had a sand batcher in place of an inundator and required continual checks on the moisture content of the sand which later were unnecessary.

Mixing and Handling Concrete.—In the first plant, sand and stone were received by truck and raised on separate bucket conveyors to the elevated sand and stone bins over the mixer which were covered to keep in the heat and keep out the rain. From these bins the aggregates fed by gravity through sand and stone batchers, where they were separately measured, to the charging hopper of the 1-yd. mixer. Cement was elevated on a belt conveyor to the platform beside the charging hopper and there handled by laborers into the hopper. Water was measured in a tank accurately calibrated. Concrete discharged direct into a two-way floor hopper capable of holding somewhat more than one batch. From the floor hopper it was wheeled in 7-cu. ft. carts to forms. With the arrival of the inundator the sand batcher was replaced and the water tank changed to a much smaller size since most of the water enters the mixer with the sand. A high pressure boiler furnished steam for heating the water, before it entered the tank or inundator, to a temperature varying between 120 deg. and 160 deg. F. This boiler also furnished steam to coils in the stone bin which extended into the sand bin where they were perforated. With the usual operating schedule this gave fresh concrete a temperature of 75 deg. to 85 deg. F. at the floor hopper.

The water supply for the inundator was too small at first and delayed the work but this was soon changed to an adequate 2-in. line. In later plants of the Mid-Continent company, inundator layouts provided a second tank for filling the inundator rapidly. Also heat was not needed in summer. All the other plants (they had six plants in all) introduced a belt conveyor to carry the fresh concrete from a batch hopper at the mixer discharge to the two-way floor hopper at slab level, which discharged into carts as before.

The use of separate bucket conveyors for the sand and the stone saved a lot of accidental mixing of the two which occurs when a single conveyor with a swinging chute is used. Steam was not allowed to condense in the stone bin. The use of a belt for elevating the concrete gives an easy check on the consistency. With our materials a visible separation would occur

at the edges of the belt when the slump exceeded 4 to 5 in. The one drawback to this concrete belt occurred when the hopper man would permit the floor hopper to empty so that the concrete from the belt fell onto the metal, causing the stone to bounce. This was eliminated by always keeping the floor hopper partly full.

Underground Construction Co. also provided a good plant, but used the single bucket conveyor for both sand and stone. Working with pretty stiff consistencies we had undersirable changes in consistencies when sand fell into the stone bin or stone into the sand bin, as would occur to a small degree with this arrangement. In this plant the cement was loaded direct from the truck to the charging hopper platform and the fresh concrete was elevated in a full-batch tower bucket to the level of the forms.

On the substructure of Sections 11 and 12 and the superstructure of Section 12, for which M. E. White & Co. had the concrete subcontract, sand and gravel were dumped in separate piles and elevated by a clamshell bucket to a two-bin tower. The inundator and the gravel batcher were mounted at the bottom of these bins and discharged into "grasshoppers" (Ford chassis on which a 1-yd. tilting bucket was mounted). The stone was first dropped and then the inundated sand. The grasshopper then drove to the mixer located at any convenient point for concreting and discharged into the charging skip of the mixer. Here the cement was added, the skip raised and the extra water supplied from a tank on the portable mixer. Carts were used to transport the concrete for short runs and other grasshoppers where the trip was longer. This was a satisfactory arrangement provided sufficient care was used by the clamshell operator and truck drivers to avoid mixing the sand and gravel and by the "grasshopper chauffeur" to avoid too rough paths.

As modified for superstructure work, the grasshopper traveled about a block to discharge into the charging hopper of a stationary mixer where the cement and lime were added. A tank on the mixer supplied the extra water and a tower bucket carried the concrete to the upper level where it was handled by carts. From the standpoint of producing high quality concrete this arrangement was satisfactory, except for the scattering of the various operations and the need of added inspectors to cover so large a territory.

Conveying concrete by means of a gravity chute was prohibited by the specifications except for very short distances (10 to 20 ft.) to reach the far side of large footings, etc. While I am willing to admit that good concrete can be and has been conveyed to place by chutes, I never have had the good fortune to see it done. A floor hopper will serve to reassemble partially the segregated materials but the uniform concrete of the mixer discharge is lost. When using a chute system the temptation to guard against a blockade in the chutes, due to an occasional over-dry batch, is too great for the good of the work. I am informed by several of the largest concrete placing organizations that no economy is effected by the use of chutes on their work and that they have discarded very expensive

chute equipment as a means to reduce their placing costs. I cannot conceive how concrete discharged directly onto heavy reinforcing steel after a long run in the chutes can ever be uniformly dense and strong. I have been called in on a few desperately sick structures where the evil effect of chute placing was the major ailment.

While I am quite ready to agree that the highest quality of concrete can be made without the inundator, we have no regrets that we required its use on Wacker Drive. Production of concrete which is to be kept constantly, batch after batch, hour after hour, at the optimum consistency depends on several things: (1) keeping the materials (cement, water, sand and stone) constant in character and grading; (2) keeping the materials constant in volume; (3) mixing them an adequate time in a mixer properly designed to mix thoroughly and discharge uniformly the entire batch, and (4) preventing segregation in transportation and placing. The inundator comes in under (2) above and is the best device I know of for insuring the uniform measurement of the sand and the water. Without an inundator resource must be had to determinations of the moisture in samples of the sand. Operating in all kinds of weather, as we did on Wacker Drive, such a method would have been unreliable and expensive. The inundator requires a workman who is patient, methodical and faithful. The opposite type can nullify the possible benefits of inundation by slopping water into the mixer or by failing to inundate fully each charge of sand. A machine would be far better than a man and I trust that some day the bulk of our concrete may be made almost entirely by machinery in central mixing plants with some single real concrete expert in complete command of each plant.

Time of mixing is another item in which practice has lagged behind the evidence of carefully conducted tests. We specified and enforced a 1½-min. mix. Rough job experiments showed that 1½ min. was not excessive when using a 1-yd. mixer and 6 to 7 sacks of cement per batch. With more cement a longer mixing time was clearly indicated and a 1: 3½ mix required 2 min. Thoroughly mixed concrete discharges more uniformly from the mixer, has less tendency to segregate and has greater strength as a result of its longer mixing. Time of mixing will also vary with different mixers and will be affected by such items as the inundator, the admixtures used, the desired consistency, the fineness modulus, grading of the aggregates, etc.

INSPECTION AND PLACING.

Value of Inspection.—Having drawn up careful specifications and carried out preliminary tests to determine the construction processes and make possible high-quality concrete, the Board of Local Improvements proceeded to effect an adequate inspection to make the results certain. To lack of intelligent supervision and adequate inspection many of the ills of concrete may be attributed. The head office of the architect or engineer is often less well informed than the field force and impairs their

efficiency. The value of good inspection accompanied by complete records was demonstrated on Wacker Drive. We had one well which was concreted under inspection to the top of the bell and left over night. The shaft was concreted at some later time without any record of inspection. Circumstances were sufficiently suspicious to warrant an investigation which resulted in the entire caisson being removed down to the bell and reconcreted. We took out 12 ft. of straight laitance most of which was quite soft after two months in place. Lower in the shaft we found large volumes of stone and sand practically without cement. The cement, water soaked and dead, had risen to the top as concreting went on. While, so far as can be learned, this concrete was mysteriously placed without the knowledge of anyone connected with the job, the facts disclosed by the removal of this caisson lead me to believe that the shaft concrete was probably dumped into 15 to 20 ft. of water which retained and floated the cement while washing the aggregate almost clean.

Caisson work was carried on continuously, working in three eight-hour shifts. The same practice was adopted for the superstructure concrete to avoid construction joints. Each half block between expansion joints and each street intersection, also bounded by expansion joints, was poured in a single continuous operation, the time required varying from about 20 hours on the smallest areas to 96 hours on our largest. Inspectors were assigned to three shifts, being rotated from week to week to even up the night work. Each shift consisted of six men as follows:

1 Inspector in charge of the shift, who determined the water tank setting, and controled the placing and mixing generally.
1 Inspector to check the mixing time.
1 Inspector of reinforcing steel to prevent any misplacement of bars in the final structure.
1 Inspector to make slump tests and cylinders, to check aggregates, and to keep temperature records.
2 Inspectors on concrete placing.

In addition either Mr. Evans or I was present most of the time—a thing that helped in several ways. Our inspectors were more on their toes and the contractor was decidedly more alert, since we were free to happen in everywhere and anywhere at most unexpected moments. Several of our inspectors were men of long experience on concrete road work (generally about the best training an inspector can have) and the rest were largely men of design experience recruited from the office of the Board of Local Improvements. The field experience gained here will make them better designers and they made very good inspectors. A few men failed to rejoice over turning night into day regularly and were excused, but most of them were cheerful and efficient. The "prize" inspection job and the most difficult to man, was the occasional excursion to the mill for a day and a half or more while cement bins were being filled for our use. No one cared for

that job, but it was loyally performed like the rest. The *esprit de corps* of Mr. Evans' office in the Board of Local Improvements was certainly high and the inspectors took pride in their responsible connection with a work much in the public eye. An engineer of tests and inspection handled the various routine proportioning tests and re-checks, assigned the men to shifts and was in charge of the cylinder curing, testing and records. The cylinders were broken at the laboratory of the Chicago Bureau of Public Works.

Care in Steel and Concrete Placement.—At the time that each concreting operation (on superstructure) started, the steel setting had been carefully checked as it went in and the cleaning work was either completed or well along. The columns had been poured in one operation and the column capitals in another before the slab steel was set. The inspection on this work was exceedingly good and the slogan of our work "every batch of concrete must be good and every bar firmly supported at least 1 in. from the surface" was carried out with surprising success. No one will claim that *every* bar was kept safely covered with a full inch of concrete, but I am satisfied that not enough are misplaced to endanger the permanence of the structure. Nor was every batch of concrete mixed a full 90 sec., nor kept within a 5-in. slump, but the whole performance was extraordinarily good. We had to replace a few mixer-timers for failure to perform satisfactorily a tiresome and dirty job. We had to remove a few contractor's men who insisted on "wetting up" the concrete and also a few foremen who were more concerned with getting wetter concrete than with placing what we gave them. The right to remove workmen who will not honestly attempt to conform to the requirement of stiffer concrete should always be included in the specifications.

Spans were long and loads were heavy on Wacker Drive and the reinforcing steel was heavy. As compared with an ordinary flat-slab factory floor with 20-ft. square panels and 200 lb. per sq. ft. live-load, our steel was 2¼ times as heavy per sq. ft. and the difficulty in placing concrete was unusual, especially at column heads. Here the steel was over three times as heavy per sq. ft. as in the factory floor cited above. The temptation was very strong to wet up the concrete and then poke the stones through the tight mesh after the mortar had run in. It was the inspectors' job to check any such tendency, and the slump record shows that they lived up to their responsibility. It was slow work and concreting had to be well organized. The number of workmen required was higher than is usual in concrete placing and the labor cost of placing was undoubtedly increased on account of the congested steel necessary in such a structure. On a front 138 ft. long we commonly had 10 to 12 spaders and delivered about 55 yd. of concrete per hour with two 1-yd. mixers operating. In bad mats our inspectors were the equivalent of extra spaders in their zeal to make sure that every bit of space was packed full of concrete. In the open panel centers, where the main steel was at the bottom and only temperature steel at the top, a workman wading around in the plastic con-

crete effected very satisfactory "spading" at very small cost, and most of the spaders worked continuously at points where the steel was heavy.

In the arched beams along the river front, where depths exceed 7 ft. in places, the use of sloppy concrete would have been inadmissible on account of the shrinkage and settlement. On Section 9 we had numerous beams, heavily reinforced and equally deep, where the use of stiff concrete was likewise essential. Some spans ran up to 45 ft. and more, requiring large volumes and multiple layers of compressive steel, and here, too, stiff concrete was clearly indicated. In general, the places where the temptation was greatest to use sloppy concrete to get it into place cheaply, were precisely the spots where wet concrete would have been more dangerous. Out at the panel centers we had no need to use such concrete, as stiff concrete was easy and economical to place there.

Our inspector in charge of each shift controlled the setting of the water tank at the mixer. No contractor's man was permitted to touch it except under his direction. At first the mixer man was not allowed to do even that much, but our man handled the water exclusively, making slight changes as minor variations in the aggregate, in the weather, or at the forms required. We found that on warm days it was necessary to allow extra mixing water to keep the desired water-cement ratio at the forms. As between a near mixer and a far mixer, both using the same materials, the far mixer required more water. As between day and night work, the day concrete on warm or windy days required more water. Since such changes were indicative of loss of water en route from the mixer to the forms, extra cement was not required to balance the extra water. Where a change in aggregate grading occasioned the extra water, extra cement in proportion was required. The inspector in charge of a shift must obviously be a man of experience and sound judgment.

Weather Effects.—We worked in all kinds of weather. Through rain, hail, snow and blizzard, the work, when once started, went on without halting. These conditions raised many questions not commented on in any descriptions that I have seen of work with scientific concrete. We made a good many tests of the effect of such water increments received from the weather on the strength and weight of our concrete. For light rain conditions we found it sufficient to use a stiffer consistency at the mixer. For heavy rain we added from one-half to one sack of cement extra per cubic yard of concrete. In some very heavy downpours specimens cut from the concrete in core boxes showed one sack of cement extra to be adequate to maintain both strength and density. In such situations the stiff concrete was a great advantage, as it resisted penetration and the rain largely ran away. We protected the fresh edge with tarpaulins to prevent washing when any was indicated. Sidewalk finish covered with brick sand immediately after the final troweling was not injured by rain.

In times of rain we were forced to rely more than ever on slump measurements of concrete at the forms. The stone became variably wetter, and in extreme cases we worked at the limit where no mixing water was needed

above that introduced by the stone and inundated sand. We did not have occasion to remove water from the batch. This was probably because material bins were roofed and turned aside water that would otherwise have found its way into the mixer. I remember one blizzard on the exposed river front, starting with a cold rain and turning to snow, during which I continued on the firing line for 28 consecutive hours. Live steam was used to clear away the snow. We made many special test specimens, including core boxes 24 in. x 48 in. x 12 in., to obtain full information as to the effect of the storm but even under these extreme conditions the precautions were adequate and the concrete tested out remarkably uniform. This storm occurred during a large placing on Section 10 on Jan. 4, 5 and 6, 1926. The seven-day tests showed a minimum of 1,711 and a maximum of 2,723 with an average of 2,273 lb. per sq. in. and 96 per cent of all tests fell within 20 per cent of the average. Eighteen sets of test cylinders were taken with seven or eight cylinders besides the core box to a set, the extra sets being concentrated in the stormy period.

Our regular winter mix of 7: 14: 22 was in use at the time and our maximum addition in the worst rainfall was 1 sack of cement per yard batch. Concrete was heated and our test specimens were stored within the enclosure on high shelves. This illustrates well the basis for our general conclusion that concreting may safely be continued regardless of weather if proper supervision and inspection are available at all times, and that no considerable extra expense is involved, at all commensurate to the loss involved in discontinuing operations, and putting in a bulkhead in heavy steel.

In stormy weather the *morale* of the contractors' forces was tested and it generally proved to be excellent. A few workmen who *had* to protect themselves did so, but the majority stood by the contractor's superintendent who cheerfully assumed every hardship asked of the men. While the men were thoroughly out of sympathy with our efforts for stiff concrete at first and while some of the key men took it upon themselves to tamper with our consistency in the early work, they soon learned that they must either conform or leave. This lesson the workmen must learn at the very beginning. It is equally important, however, to instruct the men as to the reason for severe inspection and complete control. When a man, whether he be a mixer captain or spader, was told to do a thing differently he was told why, in words and figures that he could readily understand. The workmen on the drive came to take pride in their work, which was an absolute requirement for the best work and for the success of our own endeavors.

Sand Peculiarities.—Aggregates were inspected regularly and sampled frequently. Night inspection was no less exacting than the rest. The majority of our rejections of stone were made at night and because of high dust content. Our Beloit sand, in the proportions determined for our work, gave a smooth working concrete. Another sand, of which a few loads occasionally appeared until headed off by rejections, showed in our tests

very close to the same fineness modulus, but was made up of slightly more of the finest and coarsest particles and slightly less of the medium sizes. The differences in the percentages retained on the various sieves was slight—hardly over 3 to 4 per cent on any sieve. But the difference in effect on the concrete workability of these two sands of the same fineness modulus was strikingly marked—the spaders would call it to our attention almost the moment it appeared. The intruder gave a concrete which *resisted* placing as compared with our regular sand and we had to resist its introduction on the work. This incident, which was verified by repeated experience, seems to indicate the soundness of Professor Talbot's view that the mortar-making qualities of the fine aggregate are more important than the fineness modulus.

FIELD TESTS.

At first our proportioning tests were made at the Board of Local Improvements laboratory several miles from the work. As concreting began, we took over and attempted to wrest from the rats, the basement of an old produce building but this proved thoroughly unsatisfactory in December. From here we went to another basement in which the heating plant of the building kept things comfortably warm. We were thrown at once into the very midst of a lot of questions that are rarely mentioned in papers on scientific concrete control. We hear much of the desirability of carrying a concrete operation uninterruptedly through the winter months, but what of the cylinders? Assuming that one can find or build a place where the cylinders can be stored in damp sand at temperatures favorable to curing, what relation exists between such cylinders and the concrete in the work? For such conditions two distinct sets of cylinders are desirable—one a *quality* set which will indicate the strength of the concrete when cured under standard conditions and the other a *job* set that will be a measure of the strength in the work. We found neither set easy to handle at first. Available storage places were cold during parts of the day and even when the air warmed up the damp sand would absorb little heat.

Field Laboratory Established.—We were forced finally to build a field testing laboratory which we moved as the work required and in which we installed automatic registering thermometers in the sand. This laboratory was equipped with the necessary apparatus for our routine and proportioning tests including:

1 250-lb. scale, divided to 0.01 lb.
Several black iron bread pans to use on scale.
1 nest of standard sieves, No. 100 to ¾ in. sizes, inclusive.
4 large screens of ⅜ in., ¾ in., 1½ in. and 3 in. sizes.
1 one-cu. ft. iron measure.
1 one-fifth-cu. ft. iron measure.
1 ten-in. mason's trowel with point cut off.

1 standard slump cone, with tamping rod.
Several covered metal boxes for aggregates.
Flasks, chemicals and pan for colorimetric and decantation tests.
Several glass graduates, divided in c.c.
2 mixing pans with smooth surfaces throughout and with turned-up edges.
30 metal cylinder molds, 6 in. x 12 in.
Supply of cardboard cylinder molds for core boxes.
60 machined base and cap plates.
Oil and brush for oiling and cleaning molds.

We had foot rests attached to the slump cone to permit of one-man use. Our crushing tests and our cement tests were made in the regular laboratory. A stove furnished heat and drying for aggregates while the damp sand storage for the cylinders while curing was provided on shelves arranged to permit of free air circulation on all sides. Earlier, with our sand storage on the floor and against the wall, we found that we were not getting better than 44 deg. to 50 deg. F. in the sand at any time of day (although the room temperature rose to 80 deg. F. occasionally), which was not a satisfactory curing temperature for "quality" cylinders. Standard C31-21 of the A. S. T. M. seems to me very deficient in this respect in that it ignores completely the storage temperatures, which are exceedingly important variables. Tentative standard C39-21T leaves the wide temperature range of 60 deg. to 75 deg. F. for the storage condition. Quite a wide variation in concrete strength occurs in this temperature range; as much as 700 lb. per sq. in. with our concrete.

Our "job" cylinders also required the development of new methods. At first we simply divided our cylinders, putting part in damp sand in the field laboratory and the rest on the ground under the slab placed at the same time. We found that these cylinders were not fair samples. The radiating surface of a cylinder is so much greater in proportion to its bulk than is true of the slab, that the chemical heat of setting is far less effective in the case of the cylinder. And the temperature on the ground is much colder than up where the slab is. To equalize conditions we built shelves near the top of the forms under the slab, but not over salamanders, and stored our job cylinders in this manner for one winter and for part of the second.

Best Job Curing.—Finally we hit upon our best method which is recommended for consideration. In an opening in the slab where sidewalk lights were to be placed later, we built a box 2 ft. wide by 4 ft. long by 1 ft. deep. In this we placed eight paper molds and filled these cylinders with concrete in the regular standard way. Then we filled the spaces around the molds with concrete. This gave us a considerable block of concrete, of thickness comparable although not equal to that of the slab and placed at the same level above the salamanders as the slab. The top was covered with hay and was wet by nature or by man whenever the slab

received water. When 7 days had elapsed we cut out two cylinders for test. Both ends must be capped before testing. We found a tendency for the lower end to become convex, a tendency which I have cured in my use of this method on other work by putting a thin steel disc under the center of each mold. At later times, as information was desired, other cylinders were cut out and tested. These "core box" cylinders have given consistent results, higher than was found for cylinders stored on shelves under the slab but much lower than for quality cylinders cured under standard conditions.

If it were practical to insert molds in the slab, to be later removed for tests as cylinders, as was done in one building, I should consider it a better method still, but such a program was not practical in a heavily reinforced slab like Wacker Drive. This core box method is the best we have found, although it seems reasonable that the slab concrete will be stronger than the core box cylinder, on account of its greater depth and lateral extent. Under a covering of hay, we have found satisfactory curing temperatures on the top of the slab and somewhat lower temperatures on top of the core box. A temperature of 60 deg. F. is not far from an average temperature for the slab and 5 to 6 deg. less for the core box. Where the hay was a foot or more in depth, temperatures up to 75 deg. and 80 deg. F. were found on top of thick portions of the slab. As shown in Table I the strengths in winter were satisfactory at 7 days. In fact they were generally greater than 7-day strengths in warmer weather. Twenty-eight-day strengths showed but little gain, (500 to 600 lb. per sq. in.) as the heat was discontinued after 7 days. It must be remembered, however, that our winter concrete had an extra bag of concrete per cu. yd.

Protecting Curing Concrete.—The heat applied to the materials in the making of concrete is rapidly dissipated in handling. Concrete leaving the mixer with a temperature of 80 to 85 deg. F. would show not over 65 to 70 deg. F. by the time it was placed in the forms on a cold winter day (air temperature 10 to 20 deg. F.) and would fall as low as 50 deg. F. by the time it was firm enough to cover with hay to prevent further heat losses. Tarpaulin enclosures, made and maintained with great care will show very low efficiency in keeping in the heat of the salamanders. Inasmuch as concrete in setting generates ample chemical heat for its own proper curing, it would seem that some practical method might be found to insulate the freshly placed concrete effectively against heat loss and that this might prove far more economical than the salamanders and tarpaulins enclosures now generally used. The salamanders have a tendency also to reduce the moisture below that required for favorable curing conditions. On a portion of Section 3, under moderate outside temperatures (from 25 to 35 deg. F.) it was found that covering the slab with hay, including the ceiling as well as the top, gave curing temperatures of from 65 to 75 deg. F. which were obtained at very little expense and were maintained without extra cost far beyond the specified 7-day curing period.

Data on Test Cylinders.—In the routine testing work reports were prepared of each set of cylinders and of each core box made. One set was made for each 8-hour shift and consisted, as finally adopted, of one core box containing 8 cylinders embedded and of seven cylinders cast in steel molds of which three were cured under the slab, and four in the field laboratory. This assured us of samples of at least two batches of concrete from each shift. Occasionally two such sets were made in one shift. The extra set was made in order to get samples when the concrete was less stiff than usual in order to get data on the weakest concrete in the job. Over 4,000 cylinders were taken in this way in the course of the work. Table I gives a summary of the results in the case of the cylinders representing the concrete in the upper roadway slab and gives job strengths—not quality strengths under standard curing conditions for the reason that we were not able to maintain standard curing conditions in our field laboratory with the facilities available. The table gives the data of 295 sets of field tests involving about 2,000 cylinders. While the strengths are quite satisfactory for the mix and time of year as noted, the uniformity of the tests was not exceptional. For the 25 groups of 7-day tests 83 per cent of the cylinders gave strengths within 20 per cent of the group average and 8 groups had all cylinders within 20 per cent of the average. For the 25 groups of 28-day tests, 90 per cent of the cylinders gave strengths within 20 per cent of the average while 9 groups had all cylinders within 20 per cent of the average. Considering the observed variation in the strength of the cement (tested as briquettes in tension) these results are about what should be expected with careful work under the widely varying field conditions The strengths of concrete specimens taken from caissons, retaining walls, dock walls and foundation girders were generally higher since in massive work stiffer concrete was used.

Density of Concrete.—In selecting mixes for field use the proportions giving maximum strength in the preliminary series were *not* selected but density and shrinkage were given much weight. The weight of concrete obtained indicates that this effort was well worth while. The constituent materials of our sand and stone were of normal weights (166.6 lb. per cu. ft. in each case) for Chicago limestone and sand. The high unit weight of the concrete, ranging from 149 to 155 and averaging 152 lb. per cu. ft. as compared with the usual assumed value of 144 lb. per cu. ft. indicates that a dense concrete was secured.

Conclusions from Test Results.—In order to investigate the yield, strength and density of our concrete under varying consistencies, we made four blocks each containing an entire batch from the yard mixer and placed with less spading than the slab received. Slumps of ½, 1½, 3 and 7 in. were used in the successive batches. The results are shown in Fig. 3. The data are meagre, but the indications are consistent with our numerous tests of smaller specimens. The batch with 1½ in. slump showed the greatest density or weight, but the greatest crushing strength was shown by the stiffest concrete, even though this was of a *crumbling* con-

sistency, not suitable for placing in heavily-reinforced work. A very large falling off in both strength and density was shown by the batch made with a 7 in. slump, amounting to 800 lb. per sq. in. in strength and to 5 lb. per cu. ft. in weight of concrete. This showing also is consistent with our regular tests. The use of 7, 8 and 9-in. slumps, while an improvement over old time abuses, would appear nevertheless to throw away a good deal of the strength and almost all the gain in density that we were able to realize by working with stiff concrete.

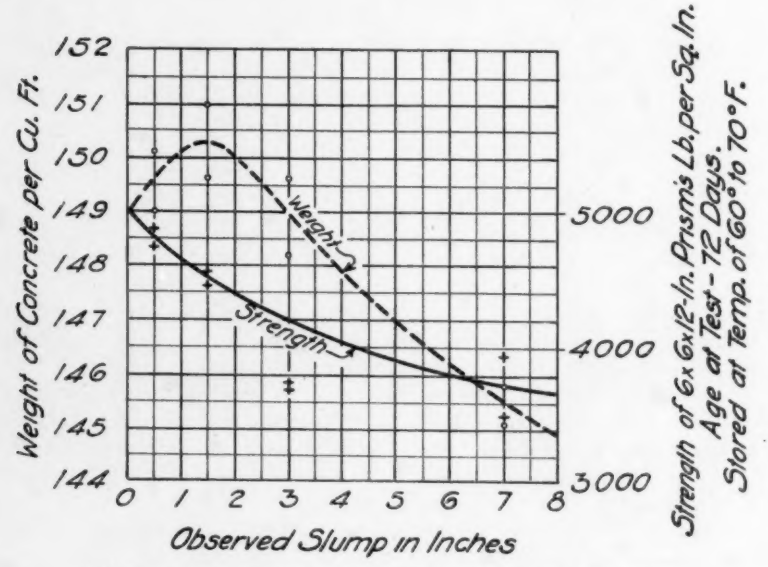

FIG. 3.—EFFECT OF CONSISTENCY ON WEIGHT AND STRENGTH.

Four full batches from the 1-yd. mixer were made into large cubes. The observed slumps were ½, 1½, 3 and 7 in., respectively. Sawn prisms from these cubes were tested with the results shown.

By comparison with Abrams' general water-cement-strength equation, $S = \frac{14{,}000}{7^x}$, the test results from Wacker Drive show very high. With from 7 to 7.45 gal. of water per sack of cement our 28-day strengths ran from 3,700 to 2,900 lb. per sq. in., while the equation would give 2,300 to 2,000 lb. per sq. in. The excess is approximately 50 per cent. The temperatures of curing ran from 59 deg. to 74 deg. F. and do not account for the excess. My own conclusion from these is that the water-cement-strength relation is in fact a related series of curves varying with different cements and aggregates and with different gradations and combinations of aggregates, and covering a zone on the Abrams graph. The particular aggregates, gradation and proportions worked out for Wacker Drive gives an

Abrams curve which lies near the upper edge of the zone and represents a highly efficient and economic use of the available materials in the Chicago market.

With our winter concrete, with from 5.9 to 6.25 gal. of water per sack of cement, we secured strengths ranging from 3,760 to 2,460 lb. per sq. in., as against values from the equation of 3,000 to 2,700. But here the curing temperature after the first 7 days was only 35 deg. F. on the average, at which concrete gains little if any in strength. With normal curing temperatures for the last 3 weeks these sets also would far exceed the values determined by the equation. The work on Wacker Drive brings home forcibly the major effects of temperature (when accompanied by moisture) on concrete strength, an item not involved in the Abrams equation since his tests were made under laboratory temperatures of 65 deg. to 70 deg. F.

Seven and 28-Day Test Relation.—We relied on 7-day tests in many of our proportioning series. This was partly because of the pressure of time limitations and partly because the relation of 7-day to 28-day cylinder strengths appeared to be consistent. Slater's equation* $S_{28} = S_7 + 30 \sqrt{S_7}$ (where S_7 is the 7-day cylinder strength and S_{28} the 28-day strength) was practically always satisfied or exceeded in our spring and summer field tests and in our laboratory tests. At summer curing temperatures of from 65 to 75 deg. F. we found this equation very reliable. Our test data confirm ordinary common sense at other seasons. In the spring when 7-day average temperatures are lower than 28-day average temperatures, the 28-day strength is much higher in relation to the 7-day strength than Slater's equation would indicate. In the fall and winter the 7-day average temperature was normally higher than the 28-day average temperature and sometimes very much higher on account of artificial heat during the first week. The 28-day strength at these seasons is much less relative to the 7-day strength than Slater's equation indicates. In April and May 7-day tests are relatively low while in September and October they are relatively high, as compared with the 28-day tests.

Importance of Temperature.—This brings us to the fact that Abrams water-cement-ratio-strength equation, even for a single combination of aggregates, is based on a temperature maintained fairly uniform in the curing room during the full test period. The same is true as to Slater's equation expressing the relation between 7 and 28-day strengths. Both are derived from data safeguarded against variation by curing temperatures held quite close to 65 deg. or 70 deg. F. The contractor in the field cannot count on this. In temperature changes, which are largely beyond his control, he has to deal with a variable as great in its influence as water-cement ratio and as neglected today as that ratio was ten years ago.

We found on Wacker Drive that one degree change in temperature was roughly equal to 50 lb. per sq. in. change in strength at 28 days. With weaker concrete I should expect a less effect. As between 75 deg. average 28-day temperature (July) and 55 deg. (May and October) this means a

* *Proceedings*, American Concrete Institute, 1926, p. 437.

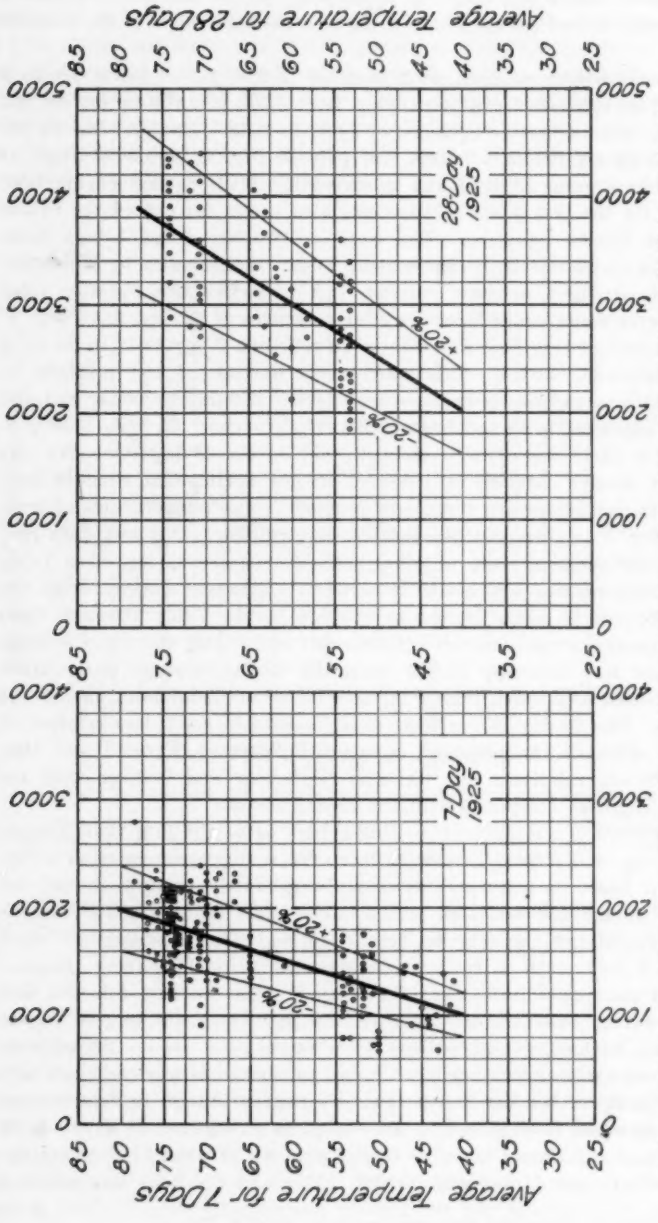

FIG. 4.—EFFECT OF TEMPERATURE ON STRENGTH: 5-IN SLUMP (1925).

loss of 1,000 lb. per sq. in. in strength. No precautions are ordinarily taken in the field at this time of year and yet, if our showing is at all generally applicable, here is a variation in strength equal to a change of 0.15 in the water-cement ratio or of 1 gal. of water per sack of cement. Furthermore, our 6 and 12-month tests, while too few in number to be at all conclusive, indicate that concrete placed under cool temperatures in April and May never catches up with that placed under mid-summer temperatures. Our whole study of the problem is inconclusive for the reason that our data were all that of field tests with all the variables operating at once. Our tests do indicate the need of adequate laboratory tests under controled temperatures to investigate this variable. Some of our data are given in Fig. 4 and Fig. 5 in which 1925 concrete with 5-in. slump and 1926 concrete with 3-in. slump are plotted, respectively. The mix was 1: 6 throughout, the water content varied from 7 to 7.2 gal. per sack of cement although the slump was constant, and the strength varied as shown.

New Form of Strength Equation.—While the graphs show the usual broadside effect that occurs wherever slump is plotted directly or indirectly against other variables it seems to me that the trend of the diagrams, the slope of the zone in which the data fall, is clearly and consistently indicated. These specimens received no artificial heat, being made in April, May, June and July, 1925, and between May and September, 1926. Other graphs have been drawn in which the calendar date is used as the abscissa and curves of cylinder strength and of average temperature are plotted. In general the strength curve is found to rise and fall consistently with the temperature curve. A warm week in May would throw the strength up, a cold week in June would throw it down again. Here also the data are too meagre, the days on which large quantities of concrete were placed too scattered, to afford anything more than a general trend. If our data could be considered adequate in amount and range to permit of writing an equation we should have, *for our materials*, the relationship:

$$S_{28} = \frac{200\ T}{4^x} \text{ in which}$$

S_{28} is the probable 28-day cylinder strength.
T, the average curing temperature in degrees Fahrenheit, and
x, the ratio of water to cement by volume.

The sole interest in this equation lies in its new *form*. The value of the various constants remains to be established and some of them will vary with different materials. To be general, Slater's equation will need to be recast in a similar manner with proper weight given to temperature in the 7 and 28-day periods. Data are needed to determine not alone the constants for different temperatures maintained uniformly for 28 or more days but also the value of heat (always with moisture) received in widely varying degrees during the 28-day period. Our data suggest the conclusion

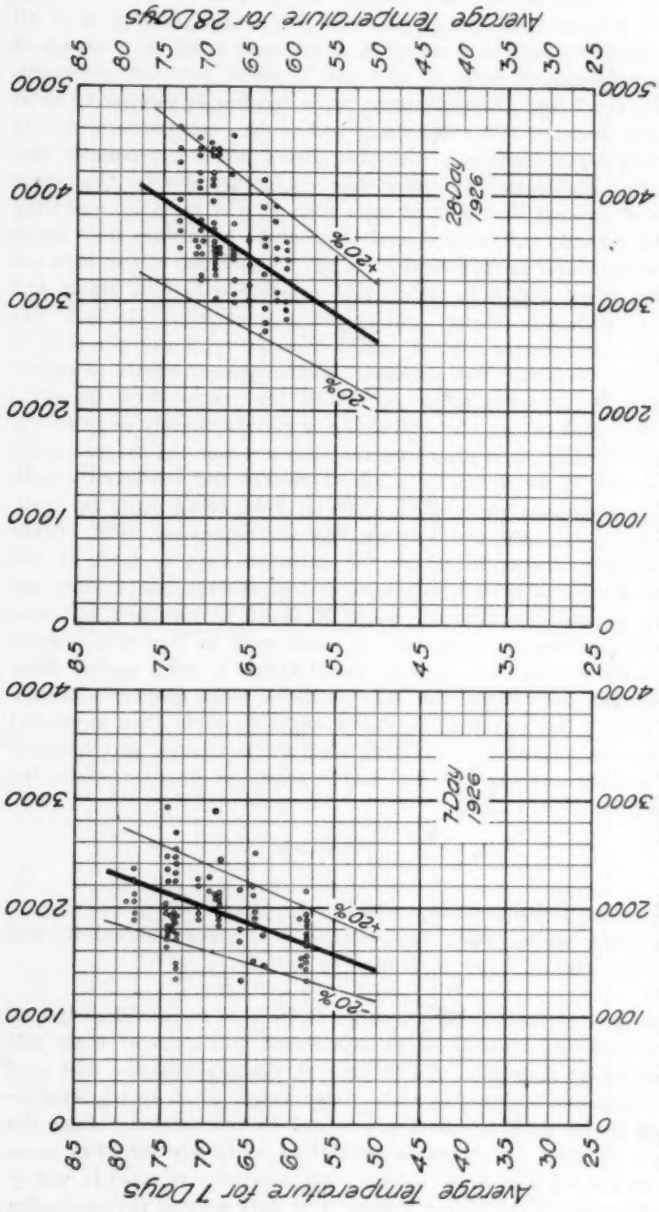
Average Temperature for 28 Days
28 Day 1926
+20%
-20%
7-Day Strength of 6x12-In. Cylinder.—Lb. per Sq. In.—28-Day Strength of 6x12-In. Cylinder.
All Temperatures stated in Degrees Fahrenheit.
7 Day 1926
Average Temperature for 7 Days

FIG. 5.—EFFECT OF TEMPERATURE ON STRENGTH: 3-IN SLUMP (1926).

that heat received during the first 7 days is much more valuable in producing strength than equal heat applied in the third or fourth week. If the dissipation of the uniformly distributed internal chemical heat of setting is prevented during the first days when it is most actively generated, the increase in strength will naturally be much greater than is the case when this chemical heat is lost and an unevenly distributed external heat is applied later. Our tests do not prove this—they suggest it only.

Temperature Often Ignored.—While the experiments of Daniels made at the University of Illinois in 1914, and reported in Bulletin No. 81, University of Illinois Engineering Experiment Station, indicate temperature effects on the strength of concrete similar in character and percentage to those found on Wacker Drive, it appears to be general practice to ignore temperature on building construction. The latest bulletin (1927) from the Portland Cement Association gives Daniels' result, but it does not adequately warn the builder of the danger. It leaves the busy contractor to dig into laboratory results in order to realize the danger involved in ignoring temperature, and contractors will not dig!

I was at a meeting of contractors recently in which one favorably known contractor, who is thoroughly conversant with water-cement ratio, spoke with pride of a building nine stories high erected in 38½ working days with *one set of forms;* and in October and November. It did not fall down and yet the strength of the concrete in those floors could hardly have been more than a few hundred pounds per sq. in. when they were stripped of their supports and called on to carry a new floor above. Certainly the strength of concrete cylinders stored for 28 days in damp sand at a temperature of 70 deg. F., which for this structure was close to 2,000 lb. per sq. in., is no criterion of the strength of concrete in floors heated for one or two days only.

I watch another concrete operation from my residence, where the contractor has only enough tarpaulins to cover about one-half of a story at a time. He therefore pours half a floor to-day, puts his tarpaulins up and heats with salamanders until tomorrow morning, when he removes the tarpaulins and salamanders to the other end of the floor and concretes that. As a matter of personal observation I know that this concrete gets as little as 16 hours of heat all told in some cases and no covering is provided *over* the slab in January with the temperature continually below freezing. His concrete was not placed with a 3 to 4-in. slump either. The forms have not been removed as I write, so I do not know what further chance this concrete may have to cure properly. At the trifling expense of a few more tarpaulins and of salamanders left in place longer this contractor would be safeguarded against the elements and his reputation and his investment in these floors would be protected against loss. Such concrete, permitted to dry out and sealed away from further moisture by walls and ceilings, has little chance of ever being 2,000-lb. concrete if past tests mean anything.

MISCELLANEOUS NOTES.

By placing the concrete in columns, in the column capitals and in the deck slab in three distinct operations it was possible to do a remarkably fine job of cleaning throughout this work. Concrete was not allowed to fall through the slab steel into the columns—the slab steel had not been placed at the time the columns were concreted. Where concrete is placed through the steel considerable of the mortar clings to the bars in successive thin films which dry out into a weak powdery substance which must have a bad effect on the bond. Our method largely eliminated this difficulty.

We found that once the form lumber had been permitted to dry out in the sun and open up cracks between the boards, it was practically impossible to close them up tight thereafter. Dirt would gather in the open cracks and wedge them open. By keeping the forms moderately wet at all times these open cracks had no chance to form and a much smoother under surface was obtained, largely free of ridges between the form boards.

The inundator balks at frozen lumps in the sand and also soon clogs if straw is present—two highly creditable discriminations for a machine to show, from the *engineers'* viewpoint. Straw is used in Chicago to plug small holes in the car in which sand is shipped. It had to be kept out of the inundator by letting the sand drop through a screen at the bucket conveyor discharge.

Wacker Drive concrete was dense and the main problem in avoiding water passage through the slab was involved by the construction work. At the curb and gutter line the vertical curb form had to be supported rigidly to line and level and these supports had to be left in place until the concrete was able to hold its shape safely. This meant that the wood uprights supporting the curb form were not withdrawn until the concrete surrounding them had taken its initial set. Even with great care some disturbance and cracking of the concrete occurred when these supports were pulled, and at first we found leaks at many of these spots. By requiring these posts to be pulled as early as possible and all such holes to be filled with fresh concrete thoroughly tamped into place, this difficulty was largely overcome.

Our stiff concrete set up rapidly which was a great advantage in many practical ways. The forms could be removed earlier. The sidewalk finish, also put in as a stiff but thoroughly plastic mortar, hardened quickly and cement finishers were commonly done at six or seven o'clock instead of hanging around expensively until midnight. The period during which workmen and visitors had to be kept off of the fresh concrete was relatively short. Every man who wades through fresh concrete leaves a series of holes that tend to fill with mortar exclusively and to show shrinkage cracks later. In hot or windy weather it was generally found desirable to sprinkle the concrete within two or three hours after placing. In a few hours the concrete was ready for its damp sand blanket which assured it of favorable and uniform curing conditions at once.

Stiff concrete exerts less pressure on the forms. An incident will show the practical value of this. One of the 34-in. diameter columns about 18 ft. high was accidentally concreted without the forms being banded. The corners of the octagonal forms were lightly nailed. This column has not been touched and yet I doubt if any reader will be able to find it should he seek it diligently, although the concrete was placed rapidly and thoroughly spaded.

Mortar was used on the deck slab to cover the forms. This was placed just ahead of the concrete which constantly squeezed the mortar out as it advanced, thus filling the spots where stone could not be expected to flow on account of the congestion of the steel. This is a much better method, in my opinion, than depending on the mortar to run out of the concrete to fill these spots with the chance of having honeycomb either in these spots or in the interstices of the coarse aggregate.

No attempt was made on this work to specify concrete by strength. Very few contractors could reasonably be expected to safeguard the strength and density of the concrete as we guarded them. On such work the responsibility for the necessary qualities properly rests with the engineers who have no divided interest in the matter. A strength specification would be a hard one to enforce, when placing concrete at all times of the year. It would be fairly simple to deliver a concrete which would meet a given strength specification as sampled, cured and tested under uniform moisture and temperature conditions, but the practical problem is far more complex.

A limited amount has been presented in this paper by way of diagrams or tables showing the test results. Our test data are necessarily field data and cannot from the nature of practical field work approach laboratory tests in uniformity and in value. Too many tests are published every year in which the necessary reduction in the number of variables has not been practised sufficiently to render their interpretation certain or valuable. The exact water-cement ratio *in the concrete as placed* in hot weather, or after long hauls, or in stormy weather is impossible to determine accurately and this is an essential variable to control accurately in research work. Curing temperatures are also hard to know and necessarily vary in the case of concrete cured in city streets and shaded to various degrees by adjoining buildings. Our tests were primarily designed to ascertain the quality of the concrete in the finished structure and with that we must be satisfied.

Ease or difficulty of placing good concrete in the field is greatly affected by the work of the designing engineer. All designers of reinforced concrete should have had extensive field experience—after which they would not continue to put up to the field men the puzzles which are now so common. For every problem in reinforced-concrete design there are several answers equally correct but far from equally simple to build. The closeness of the mesh formed by the bars, the pitch of the spirals, the allowance for stirrup bending and the size of bars require especial con-

sideration. Much bad handwork in the field is caused by poor headwork in the office.

The supporting of the reinforcing steel as its designed position vertically in the slab was a knotty problem for two reasons. First, the steel was very heavy, and second, we wanted to avoid unsightly rust stains. The use of galvanized chairs was tried on the first section but the weight of steel tended to drive these into the forms and they were abandoned after a single trial. Concrete blocks of various heights and sizes were used practically throughout the job. Some of these were made of lumnite cement and these proved satisfactory except for color. Portland cement with 2 per cent calcium chloride was used most of the time. For the bottom steel, blocks 1½ in. square by 1 in. high were used to hold up a ½-in. square supporting bar. This provided more than 1 in. clearance between the reinforcing steel and the forms in general. For the upper steel, supporting blocks were hexagonal in section and of many different heights on account of the drainage slopes on the upper surface of our slabs.

Lumnite cement was used in our work in several instances where speed was the dominant factor. We used lumnite cement concrete where the improvement joins into Michigan ave., a busy thoroughfare whose traffic was partly interrupted for less than two days. Here a 1: 8.3 mix with 6.5 gal. of water per sack of cement gave 1-day strengths of from 4,100 to 4,300 lb. per sq. in. This was paving concrete placed with a slump of ¼ in., very uniformly repeated. On other work using our regular summer mix of 6: 14: 22 (cu. ft. per batch), lumnite cement gave 1-day strengths of 4,300 to 4,700 lb. per sq. in., when made with a 2 in. slump and cured by flooding with water. The 7-day strength was 5,960 to 6,160 lb. per sq. in. Lumnite cement requires much more than usual care in handling. The mixer must be clean to an unusual degree. A little portland cement left in the corners, after an ordinary cleaning, was responsible for one batch solidifying before it could be gotten out of the mixer. Lumnite cement concrete must have ample curing water applied as soon as the initial set occurs and maintained for at least 24 hours and preferably for the usual 7 days. There are simple ways of making high strength concrete in very short time by using additional ordinary portland cement with less water per sack, etc. I have been unable to feel that the addition of such large amounts of extra cement can be made without serious effect on the shrinkage of the concrete.

Many engineers have no use for the slump test and I am well aware of its variability. Different operators may obtain quite a range of slump from the same concrete and failure to perform the filling of the mold in the standard way will also give different results. But we found the slump test exceedingly valuable to have on the job at all times. When the weather added or subtracted water between the mixer and the forms, the slump test was an invaluable check on the concrete and was adequate to

TABLE 1—SUMMARY OF DATA OF SUPERSTRUCTURE CONCRETE

Date of Work	Average Temperature (1)		Water, gallons per sack	Average (2) Slump	Mix (3), cu. ft.	Weight of Concrete	Average Cylinder Strength, 6 x 12 in.		Number of Sets (4) of Cylinders
	7 days	28 days					7 days	28 days	
1925									
May 10-12	52	63	7.15	4.7	6:13.8:21.5	151	1470	3450	12
June 8-12	72	71	7.0	5.6	4:9.9:12.9	152	1840	3370	13
June 19-22	68	72	7.1	5.1	4:9.9:12.9	152	1780	3710	8
Aug. 10-11	75	74	7.45	5.0	6:14:21	149	1590	2960	9
Aug. 21-22	71	74	7.1	5.0	6:14:22	150	1840	3420	29
Sept. 18-19	67	59	7.05	3.4	6:14:22	150	1870	2880	15
Oct. 1-2	57	47	5.9	3.1	7:14:22	153	2040	3760	8
Oct. 21-22	40	39	6.25	3.7	7:14:22	153	1770	3470	12
Nov. 3-4	40	38	6.2	3.3	7:13:23	(b)	1450	2460	10
Nov. 4-5	40	38	6.1	2.5	7:14:22	(b)	1750	2830	11
Dec. 4-5	30	24	6.05	3.3	7:13:23	(b)	1560	2570	11
Dec. 17-18	25	25	6.1	3.1	7:14:22	(b)	1880	2460	12
									150
1926									
Jan. 4-6	28	26	(a)	2.7	7:14:22	(b)	2270	2900	18
Jan. 6-7	26	26	(a)	2.9	7:14:22	(b)	2120	2620	8
Apr. 12-13	39	51	(a)	3.1	7:14:22	155	2050	4430	8
May 3-4	60	58	(a)	2.7	7:14:22	154	2530	5050	8
May 11-12	57	59	(a)	3.1	6:14:22	153	2260	4160	8
May 20-21	58	61	(a)	3.0	6:14:22	154	1880	3850	10
June 7-8	66	65	(a)	3.2	6:14:22	153	1840	3540	8
June 17-18	68	67	(a)	2.9	6:14:22	(b)	1790	3550	10
June 28-29	68	71	(a)	3.2	6:14:22	(b)	2030	3690	10
July 1-2	70	71	(a)	3.3	6:14:22	(b)	2020	3840	8
Aug. 25-27	73	69	(a)	3.8	6:14:22	(b)	1830	3440	27
Sept. 8-9	66	63	(a)	3.6	6:14:22	(b)	2000	3160	8
Sept. 30-Oct 1	62	53	(a)	3.0	7:14:22	(b)	2390	3590	14
									145
Grand average of all tests							1870	3360	295
Average for months—May to Oct							2000	3540	217
Average for months—Nov. to April							1890	2850	78
Average weight of concrete as measured, 152 lb. per cu. ft.									148
Average observed slump for year 1925									4.0 in.
Average observed slump for year 1926									3.1 in.

(a) The water-cement-ratio was not accurately determined on account of the water added by stormy weather on Jan. 4-6 and on a few other dates, and because of the fact that we learned by experience that water was lost on many occasions due to evaporation between the mixer and the forms. The water-cement-ratio given in this column were carefully observed at the mixer but may not be accurate for the concrete in the forms.

(b) Determination of the weight of cylinders and blocks after from 16 to 24 hours, was omitted in many cases or so few measurements were made as not to permit of publishing an average for the period.

(1) Temperatures are averages of hourly readings out-of-doors. In winter, under the enclosure made for heating the concrete, temperatures were much higher, reaching 85 deg. F. at times and dropping to 55 deg. F. at other times.

(2) Slumps were taken with each batch of concrete used to make test cylinders. In addition, numerous slump tests were taken at other times. The average given here is for all slump readings on these dates.

(3) Sand was measured in inundated condition. Stone was measured loose and damp as received. For the 6 : 14 : 22 mix, the corresponding dry-rodded proportions would be 6 : 14 : 20; while the ordinary loose-and-damp job proportions would be 6 : 18 : 22.

(4) Each set comprises seven to eight 6 x 12-in. cylinders. The 1926 sets include also one core box with 8 cylinders embedded.

save us from the danger of placing poor concrete under some very extreme conditions when the true water-cement ratio could not be determined.

We were deluged with attempts to add various ingredients to the concrete. I have never been able to understand how the average salesman makes a living (he seems to do at least as well as the average engineer) and the technique in this case was just about average. Some preparations required not over 4 gal. of water per sack of cement in mixing—which would certainly give high-strength concrete could it be gotten out of the mixer or handled in the forms. The absurdity of most of the claims was such as to make the problem of turning them all down, even though supported by excellent political arguments, much easier.

The addition of lime was specified for reinforced concrete other than lightly reinforced massive work and except in cold weather. It was not used from any faith in reported test data, where an increase in strength due to other causes has been attributed to the lime, but for practical reasons. We found that the use of 50 lb. of hydrated lime per yard batch of concrete had no observable effect on the strength of our concrete; and we have probably three thousand tests to judge from. We found also that the use of lime in this proportion made our concrete more workable, easier to place and less liable to segregation in transportation and handling. Past experience indicated that it decreases permeability while twenty years' experience in its use in this manner has not confirmed any opinions to the effect that the lime will be carried away after the concrete is once properly made and placed.

It is not my intention to condemn all admixtures for concrete. Some of them have merit. Others have the power to correct well-known job abuses and thereby improve the quality of the concrete by indirection. None of them is necessary to the production of the best quality of concrete. On public work we felt it to be the wisest policy to rule them all out. Unfortunately, the less commendable admixtures, with the most absurd claims, found the strongest political backing.

Runway dirt (spilled concrete which dries up and blows around or is shaken off the runway plank in handling) was very materially reduced in amount by the use of our stiff consistencies. There was much less possibility of slopping over. With this start, we insisted on any spilled concrete being cleaned up at once and where a main runway was to remain in place several hours we required canvas protection sheets (tarpaulins) to be placed on the reinforcing steel under such runways to catch the dirt including splintered wood. These sheets were rolled up and removed with the dirt in them as the concreting operation required.

As I reach this point in my writing (Jan. 6, 1927) I have learned of the death of John J. Sloane. Mr. Sloane, as President of the Board of Local Improvements, frequently went over the work and was keenly appreciative of the high standards maintained on the Drive. Whenever discussion arose on the job (and it arose pretty frequently at first before all the material men and contractors became converted to scientific concrete)

it was Mr. Sloane's custom to call his engineers into the office and to ask for a full statement of reasons for our rulings and decisions to which objection had been taken. His questions were searching. Having mastered the argument, quickly but thoroughly, he would call in the objectors and confound them with his expert knowledge. He was fair to the point of leniency and gave all offenders more chances than seemed nesessary to me, but when they refused to do their part he invariably supported his technical advisers.

The Portland Cement Association have been very helpful in our work, assisting us to locate inspectors of the necessary experience, and giving us freely the use of their concrete cutting saw in order that we might better investigate the interior of our concrete and determine the strength and weight of samples from the work. They have also encouraged us by commending the work to others and by refraining from any interference when our arguments happened to concern the cement produced by members of the association. For such aid on this particular work and for their invaluable research work in general, which in truth has made the quality of Wacker Drive possible, I here express my great gratitude and indebtedness. While I have expressed herein certain differences between my views and those of the eminent investigators of the association, the main difference in our points of view, as I see it, lies in the fact that I believe that the best concrete, like the best broth, requires the services of an expert, while they believe that anybody of ordinary ability can make good concrete out of almost any usual materials. I submit that there is merit in both points of view; but mine is the better!

The motion pictures which accompany this paper were taken on the work by the writer using a Bell & Howell Filmo cine camera carried in the hand. Absolutely without any previous experience in this line and with no special attainments in photography, any credit for the merit of these films must go to the fool-proof qualities of the camera. Only a minor fraction of the exposures were unsuccessful on this work.

Discussion—Notes on Wacker Drive.

Mr. Gilkey. Herbert J. Gilkey (*By Letter*).—This remarkable paper records, with the clarity of a photograph, the present state of the concrete art. It shows that lessons learned in laboratories can be successfully applied to jobs. Comparing the attainments on this work with those possible five years ago furnishes cause for satisfaction on the part of those who have labored hard to discover "concrete" facts and to apply them. The paper also shows another side. It brings out only too clearly the enormous quantity of work still involved in "building as well as we know." A vast amount of personal supervision, the education and training of contractors, superintendents, laborers, inspectors, materials dealers, politicians and other co-workers required almost superhuman effort, tirelessly sustained, on the part of Mr. Lord and Mr. Evans. This work is a monumental piece of pioneering. Practically every known laboratory fact that could contribute toward better concrete was placed in the mixer and deposited in the forms of Wacker Drive. Thus we have in this improvement a milestone in concrete development. It represents the present state of the art at its best. Because of this work others will be able to attain equally good results with far less effort. The trail will gradually become a highway.

Not least among the valuable points brought out in this paper are the numerous allusions to the things that we don't yet know about concrete. Here is one instance in which the "job" has not only caught up with the laboratory but points with an insistent finger at new problems that the laboratory must soon solve.

The temperature effect on curing is perhaps the most important of these practically unevaluated and little considered items.

The necessity for adequate moist curing and the desirability of stiff mixtures, where placing is most difficult, are both points that have too often been ignored or overlooked.

The distinction between concrete for outdoor exposure and that not exposed is most pertinent as is also the distinction between "job" and "quality control" test cylinders.

The practical man generally feels that the test cylinder should be treated just as the structure is treated and should be an index of the strength of concrete in the structure at the age of test. On the other hand, the laboratory man is likely to feel that all samples should be cured in a standard, invariable manner in order that the quality of the concrete, as mixed, may be comparable for all parts of the work. Both are right. Both "job" and "quality control" information are needed and where the size of the work warrants and the necessary conditions can be met, test specimens should be taken with these two different sets of information in mind. To get "job" cylinders that are truly representative requires not a

little care. They may dry out or temperature may vary at a different rate from the concrete that they represent. It is probable that the "job" cylinders, as well as those for "quality control" should be wet at test. The saturated condition, practically reached in 10 to 24 hr. immersion, is the only easily attainable standard condition for test. Any intermediate state, between wet and dry, complicates the strength relation to an extent that should not be ignored but which cannot be evaluated. The difference between the saturated and the dry condition may be 40 per cent or more[1] in favor of the dry specimen.

It is interesting to note Mr. Lord's comment upon the water-cement ratio strength relation. He states:

> "By comparison with Abrams' general water-cement-strength equation, $S = \frac{14{,}000}{7^x}$, the test results from Wacker Drive show very high.
>
> "With from 7 to 7.45 gal. of water per sack of cement our 28-day strengths ran from 3,700 to 2,900 lb. per sq. in., while the equation would give 2,300 to 2,000 lb. per sq. in. The excess is approximately 50 per cent. The temperatures of curing ran from 59 deg. to 74 deg. F. and do not account for the excess. My own conclusion from these is that the water-cement-strength relation is in fact a related series of curves varying with different cements and aggregates and with different gradations and combinations of aggregates, and covering a zone on the Abrams graph."

This fully accords with the findings of the writer as set forth in a paper before this convention.[2]

Under the general heading "Workability and Strength" and also under the heading "Inspection and Placing," Mr. Lord calls attention to the fact that unless a fairly stiff mixture be maintained concrete will be extremely non-uniform as placed. Recently, emphasis has been concentrated upon the water-cement ratio to the exclusion of practically all else. "Make the concrete as thin as you please, use any kind of grading of orthodox materials that you like, but maintain as constant the water-cement ratio corresponding to the desired strength." The formula is a simple one. The flaw, aside from that of cost has been very clearly pointed out by Mr. Lord. If the mixture be at all wet, the phenomenon of "Water Gain"[3] is present to a very objectionable degree. Regardless of the water-cement ratio at the mixer, the top concrete gets wetter and wetter and all upper surfaces and upper portions of members are of very inferior material, corresponding in extreme cases to the "pier that was poured, in the inspector's absence," and subsequently removed.

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[1] "Curing Conditions of Mortars and Concretes," *Proceedings* A. C. I., 1926, pp. 395-436. "Finishing and Curing of Concrete Roads," Discussion—*Proceedings* Am. Soc. C. E., Jan., 1927, pp. 79-93.

[2] "The Coarse Aggregate in the Concrete as a Field for Needed Research."

[3] "Water Gain and Allied Phenomena in Concrete Work," *Engineering News-Record*, Feb. 10, 1927.

In bodies of concrete having appreciable depth, the only way to keep the "water gain" within reasonable bounds is to maintain a very close liaison between mixer and forms. Water-cement ratio need not be varied but the mixture must be made more and more stiff by the addition of aggregate in order that the excess water of the top concrete may be taken up by the very stiff mixtures added as placing progresses.

In such a case, slump at the mixer means nothing. It is the workability, during and just after placing, that is important. Perhaps a word should also be said relative to the extravagance of excess workability even though the water ratio be kept constant. The volume of a wet batch could have been materially increased (and shrinkage, laitance, separation and attendant evils diminished) by the use of more sand and stone without alteration in quantity of water or cement. This point is strikingly brought out in the Table of Assumed Strength of Concrete Mixtures published by the Bureau of Standards.[1]

TABLE I.—ASSUMED STREGTH OF CONCRETE MIXTURES.

Water (gal. per sack cement)	Assumed 28-day strength lb. per sq. in.	Approx. mix. vol. of cement to separate volumes of fine and coarse aggregate		
		Plastic (slump 1 to 3 in.)	Moderately wet (slump 6 to 8 in.)	Very wet (slump 10 in. or more)
8.00	1,600	...	1 : 6	1 : 5
7.25	2,000	1 : 6	1 : 5	1 : 4
6.50	2,500	1 : 5	1 : 4.25	1 : 3.25
5.75	3,000	1 : 4	1 : 3.50	1 : 2.50

This table can be accepted as representing the conservative generalized conclusions of qualified engineering experts. It should speak for itself. There can be little question about the economy of using a 1 : 6 plastic mixture instead of a 1 : 4 sloppy one, or a 1 : 4 plastic against a 1 : 2½ wet. And this volume saving is over and above the elimination of all the other objectionable features of wet concrete.

Mr. Lord's contentions are also well supported by the figures of $4.38 against $6.01 per cu. yd. cited in the report of committee C 6[2] before this convention.

Mr. Lord's paper contains a wealth of practical common sense. It directs the attention to many points that we have looked at without seeing or that we have seen without comprehending. If every few years the Institute might have just such a paper as this one, concrete progress would be recorded by a notable series of milestones, not alone telling of ground covered, but pointing on down the road of progress with a large and challenging arrow.

[1] "Recommended Building Code Requirements for Working Stresses in Building Materials." Report of Building Code Committee of the U. S. Department of Commerce, Bureau of Standards. Elimination of Waste, Series BH 9, 1926.

[2] "Economic Advantages, Field Control of Quality of Concrete."

G. M. WILLIAMS (*By Letter*).—This paper is the best description of the co-ordination of laboratory studies with details of field operations which the writer has seen. It is such supervision and inspection of every detail involved in the production of concrete which will insure the quality assumed in design and tend to minimize the common and often justifiable criticism that concrete, as made, is an unreliable building material as compared with steel. Mr. Williams.

The laboratory procedure employed by Mr. Lord to determine the concrete-making qualities of available aggregates, in which the selection of the proportions is based upon a comparison of the strengths and working qualities of the concretes, seems to the writer to be a logical method.[1] The sand-gravel ratio which will produce the best concrete is usually that combination which has the greatest weight per cubic foot. Usually a variation of several per cent of sand either side of this maximum weight percentage will have little appreciable effect on strength so that the field concretes, which will differ due to variation in gradation of different lots of aggregates from the same source, will be of fairly uniform quality.

Knowing the minimum consistency or flowability that can be employed, the necessary proportions of cement, fine and coarse aggregate can be selected from the results of the laboratory tests, and it then becomes the duty of the inspector at the mixer to see that the quantities going into every batch are the established ones and that the consistency of the concrete as discharged from the mixer is not greater than the specified slump. With these factors controlled the writer can see little object or benefit in attempting to measure or determine the actual quantity of water going into the batch. So long as the material proportions are correct and the slump is that specified the concrete *at discharge* must be practically the same as that produced in the laboratory under the same conditions of mix and consistency. In the field there will usually be some variation in gradation and quantities of fine and coarse aggregate from batch to batch but this is a variable which cannot be avoided or controlled within too narrow limits. The position of mixer man is one of the key positions and unless a dependable man can be found for the place the writer would make the inspector responsible for the consistency of the water in each batch. With a little experience the inspector will be able to observe the flow of concrete in the mixer drum and control the added water so closely that the variations of flow from batch to batch will be only a few points as measured by the flow table which is considerably more sensitive in detecting consistency changes than the slump test. Accurate consistency control will require holding back a small portion of the mixing water until the mix is nearly completed and then the addition of enough to bring the concrete to the specified consistency as determined by judgment and checked by the slump test.

As Mr. Lord points out, the adherence to a constant water-cement ratio relation may result in concrete more flowable than necessary for the pur-

[1] *Journal*, Engineering Institute of Canada, May, 1921.

pose of placing. Any flowability greater than that required for proper placement in the forms with a reasonable amount of spading and tamping, results in a weaker, more permeable concrete than that which might be obtained with the same materials. It seems to the writer that over-emphasis of water-cement ratio complicates and sometimes defeats proper consistency control at the mixer and results in a less durable concrete even though the assumed strength may be attained. With proper and reasonable attention to the quantities of materials charged into the mixer greater uniformity in strength and density will result if flowability is made constant and the inspector permits the addition of only sufficient water to attain this condition. Of course such procedure assumes that proper attention is also being given to the characteristics of the aggregate and that any marked variation is taken account of by adjusting the volume proportions of cement and aggregates in accordance with the results of the laboratory tests. Where placement is being made in deep forms the consistency should be from batches already in place and aggregate volumes can be increased to maintain constant strength of concrete.

In the measurement of aggregate quantities no doubt some type of mechanical device which will automatically furnish equal weights or volumes of aggregates to successive batches is desirable. On large work where the cost of such an installation is justified inspection is made easier and more efficient and a more uniform quality of concrete should be obtained. Without detracting in any way from the value of the inundator, batches or weighing devices employed on large work it seems to the writer that the difficulty of producing a uniform concrete without their aid has been somewhat exaggerated, and that engineers on small jobs who must adhere to the older and apparently cruder methods of measuring materials need not feel that the production of good, uniform concrete is hopeless.

Assuming that concrete proportions are based upon laboratory tests in which dry, loose weights of aggregate are employed, what is the effect of moisture in sand on the field concretes?

Concrete "A" below is made up of dry aggregates, and concrete "B" is identical except that the quantity of sand in the batch is that which would be used in case the sand were saturated and bulked up to its greatest volume. Results following are from actual tests:

	Concrete "A"	Concrete "B"
Weight of dry sand per cu. ft...	100.5	87.8 (dry sand in 1 cu. ft.)
Proportions by volume	1 : 2.5 : 2.5	1 : 2.5 : 2.5
True proportions	1 : 2.5 : 2.5	1 : 2.2 : 2.5
Lb. cement per cu. yd. wet concrete	538	572
Flowability	195	195
Density of concrete	0.791	0.791
Compressive strength, lb. per sq. in.	2,405	2,790

As would be expected "B" is a stronger, richer concrete containing a smaller volume of sand than "A." Concrete "B," due to the sand being bulked before mixing rather than in the batch as for "A," contains 34 lb. more cement per cubic yard, or slightly over 1/3 bag. It is apparent that the contractor rather than the owner should worry about the use of wet aggregate. If aggregates proportioned dry will furnish the necessary strength the contractor is furnishing a more expensive and stronger concrete. The particular sand used in the above tests was quite coarse, lacking finer particles so that the bulking effect is considerably less than would result with a finer sand such as used in many localities. These results add to the evidence which concrete contractors have discovered when checking up cement used against cement estimated for a job and emphasize that bulking is a real problem which should be faced and solved by the engineer. The following tabulation would indicate that the problem is not as serious as it appears and that it can be corrected in fairness to all concerned with a little attention to the physical characteristics of sand used.

Following are the results of bulking tests on three sands in general use in Western Canada in concrete construction work:

Per cent water in sand by weight	Weight of dry sand in 1 cu. ft. Sand No. 1	Sand No. 2	Sand No. 3
0	104.5	104.6	94.6
3	85.0	87.0	72.9
6	85.5	85.4	69.5
9	85.6	85.7	68.7
12	86.3	86.8	67.8
15	88.1	94.0	69.3
18	88.9		70.9
21	94.0		72.6
24			73.0
Moisture content in pile of sand one hour after complete saturation, per cent	14.1	12.5	20.1
Variation in weight of dry sand per cubic foot between dry sand and maximum bulking condition, pounds	19.0	19.2	26.8
Variation in weight of dry sand between 3 per cent moisture and maximum moisture sand will hold, pounds	2.5	4.0	5.0

The above figures would indicate that the greatest bulking occurs between 0 and 3 per cent of water. Additional water up to the greatest amount that will be retained by the sand when piled in the open, after

rains, results in a variation of 2.5 to 5 lb. additional. This latter variation in weight per unit volume is too small to be compensated for in practice so that a correction for bulking up to the 3 per cent moisture content plus one-half of the additional weight decrease should give uniform quantities of sand for each batch.

Since wet sand supplied to the mixer results in a homogeneous concrete with fewer revolutions of the drum, or in a shorter mixing time, it would seem that if the pile of sand is kept moist and the volumes supplied to the mixer are corrected, on the basis of bulking values obtained in the same manner as the above, uniform quality of concrete will result and the cement over-run will be eliminated. For example, if proportions as determined in the laboratory are based upon dry loose weight or 104.5 lb. cu. ft. for Sand No. 1, keep the sand pile moistened and make the actual field sand volumes $\frac{104.5}{86}$ or 1.2 times the laboratory volume. No account need be taken of the actual amount of moisture in the sand except that less additional water will be needed and a competent inspector or mixerman will have no difficulty in turning out a concrete having a flowability within the specified limit.

Mr. Johnson.

C. S. JOHNSON (*By Letter*).—Mr. Lord's paper is instructive to me because we have here one more paper where the author deals with so many of the problems involved in efforts at field control of "quality" in concrete. The comments with "inundation" constantly in mind should be carefully considered and the eye of the skeptic is here justified and should be of value. Wherein Mr. Lord explains that perfection in gradation is one essential adjunct of "inundation" he confirms my own convictions which are based on logic, research and practical observations. He is indirectly calling attention to a basic fact of great importance which should be realized by engineers: The theoretical principle of "inundation" is entirely a hope of constant voids in inundated or quick sand. A hope to have constant voids makes perfect gradation necessarily one prerequisite.

The voids used by the "inundator" are not the voids existing in rodded sand which is recognized as the nearest absolute volume measurement. Likewise, they are not the voids in any specific density of quick sand. Quick sand is sand in a loose state with the voids filled with water, and is subject to variable density. Pressure, method of depositing, gradation, silt, and time all influence the density of quick sand. It is common knowledge that voids vary in coarse aggregate, in rodded sand and loose dry sand. Is nature so whimsical as to be partial to quick sand and favor it with constant voids? To expect inundated or quick sand to hold constant voids, batch after batch, is hardly in line with sound reason or with the facts cited.

Mr. Lord's comments on "inundation" are extremely brief. However, attention is called to four comments in the paper. One is: "We have no regrets that we required its (inundation) use"; this is hardly an expression of confidence or enthusiasm. The second is: "Inundation" requires a

workman who is patient, methodical and faithful." To this I ask: What could such a man do not hampered with inundation? The third is: "Measurements of sand were made in the inundator which gives dry rodded volumes." Surely Mr. Lord does not mean what he says. Or can it be he has fallen victim to an erroneous popular conception? I courteously and respectfully invite anyone to prove the latter part of his statement to an unbiased committee of recognized authorities in the Institute. If Mr. Lord has made this mistake, then both his sand measurements and his water measurements are erroneous. The fourth is: "The inundator is the best device I know of for insuring the uniform measurement of sand and water." To this I respectfully say, "We are in need of something vastly better than the present 'inundator theory' to work with." Attention is called to these four points and to well-known common comment to the effect that even ordinary water control was not obtained on the initial installation and that changes were made. I understand, and the paper confirms, that control of gradation was one change sought. It is known that means for full premeasurement of water was another change. This being the case it is clear that the inherent basic defect in the inundation theory was encountered.

Mr. Lord derives suggested views regarding the water-cement ratio principle. No doubt his suggestions are of value as general comments. However, we should know that he positively knew his water-cement ratios and in this connection one serious question arises and is not answered so far as I can determine. How did they know the amount of water used?

A. If they depended upon the "inundation" principle then the water measurements are questionable to a large degree.

B. If they depended upon the premeasurement tanks (inundation and excess water tanks) and allowed the inundator to overflow then the water measurements are equally questionable.

C. If they depended upon premeasurement tanks and did not allow the inundator to overflow they were subject to error due to moisture in sand and the usual doubt.

D. If they depended upon premeasurement tanks, did not allow the inundator to overflow and coupled moisture tests in sand and allowances with premeasurements then and only then can the water measurements be considered reliable.

Positive faith in water measurements and in cement measurements are prerequisites to faith in any comments regarding other variables in concrete. Anything that inspires a false sense of security should be religiously avoided.

Attention is called to the following basic truths. With a 1:2, cement-sand ratio, a specific gravity of 2.65, and the given water factors, with voids in inundated sand varied to give $+5$ per cent and -5 per cent

(95 lb. to 105 lb.) changes in dry sand contents the following errors in water contents are inevitable with the "inundation" theory:

Gal. Concrete	—Water Cement Error—	—Water Cement Error
5	+ 9.03 per cent	— 9.03 per cent
6	+ 7.53 per cent	— 7.52 per cent
7	+ 6.46 per cent	— 6.44 per cent
7½	+ 6.01 per cent	— 6.01 per cent

Tests for the inundation theory as reported in the A. C. I. *Proceedings* of 1923 show up to 4 per cent in dry sand content under laboratory conditions. Field variations are bound to exceed laboratory variations because of the vastly greater aggregate segregation chances.

A decrease in the intermediate sizes in aggregate gradation quite logically increases slump because it lowers the angle of repose and contributes to segregation. Serious as this may be, a far more damaging factor is simultaneously dragged in when "inundation" is used. The decrease in the intermediate sizes causes increased water "inundation." This increase in water coupled with lowering of angle of repose logically multiplies segregation. A shortage of intermediate sizes coupled with "inundation" logically results in the opposite effect on slump. No wonder engineers inspired with a sense of security in "inundation" are baffled by their slump tests. From the foregoing, one is logically justified in being very skeptical about assumptions based on "inundation" as a water-cement ratio control means.

Acknowledgment is granted of reasonable reliability of "inundation" as a process for reducing sand bulking to a degree that gives a fair control of dry sand contents and to within some 5 per cent to 12 per cent of the dry rodded quantity. Attention is called to the fact that compensated volume adjustment of standard volume measurement hoppers is giving dry sand control with an average error less than 1 per cent and extremes within 5 per cent under job conditions where moisture varies between around 2½ per cent and 7½ per cent and gradation is fairly uniform.

Valuable as Mr. Lord's comments are for future reference it seems that further information with water-cement contents positively known should be required as a prerequisite to comments as to the influence and practical effect of other possible variables in concrete.

Construction of Reinforced-Concrete Pier, Bremerton Navy Yard.

By Lieut.-Comdr. J. J. Manning (C. E. C.), U. S. Navy.

The city of Bremerton is located on Puget Sound in the state of Washington, some twenty miles west of Seattle. The Puget Sound Navy Yard is located at Bremerton and is the one and only industry of the city. The Navy Yard employs on an average between 2,500 and 3,000 men and Bremerton with its adjacent city, Charleston, have a population of approximately 10,000 people. The city of Bremerton is called the "Home of the Pacific Fleet" because all the battleships on the Pacific are repaired and overhauled at the Puget Sound Yard.

One of the first requirements of a yard carrying on battleship repair work is large heavy piers which will permit the battleships to come directly alongside and be supplied with all necessary services, i. e., steam, water, air, electricity, etc., so that the ships' entire boiler and engine room may be completely shut down for repairs and overhaul.

The pier facilities at this yard were recently greatly augmented by the completion of what is known as the repair and fitting-out pier, so named because of its adaptability to provide facilities for the construction of new ships as well as to repair existing ships.

This pier is a modern reinforced-concrete pier capable of berthing simultaneously four capital ships, is 1,318 ft. long and 100 ft. wide, of open construction, i. e., not solid fill, and has no pier sheds on its deck. The structure is protected by creosoted timber fender piles, spring corner fenders, and liberally supplied with tracks, bollards, cleats, bitts, and capstans. Outlets for various service connections to ships are located at convenient intervals at all berths, and ample duct lines have been provided to supply large numbers of electric welding outfits now so extensively used in ship repair work.

There are several unusual features of interest to the construction world and the engineering profession embodied in this pier. The pier is supported on reinforced-concrete cylinders with shafts 4 ft. 6 in. in diameter and an enlarged bell-shaped bottom 15 ft. 3 in. in diameter to provide increased bearing area for cylinders bearing directly on hard soil, and to encase the tops of pile clusters when the cylinder is supported on piles. These cylinders are located five in a bent on 20 ft. centers, with bents spaced on 30-ft. centers. Two expansion joints are provided at which points the bents are 24 ft. on centers and the deck supported on 12-ft. cantilever beams. A large reinforced-concrete cross girder caps the cylinder into which girder the reinforced-concrete track girders and floor beams are framed.

Built integral with the pier of the same general type of construction, i. e., cylinders on piles, is a foundation to support a 350-gross ton hammerhead crane, the reaction on each of four legs of which is 3,000,000 lb.

The reinforced-concrete deck is designed for uniform live-loads of 900 lb. per sq. ft., excepting that section served directly by the 350-ton crane which is designed for 1,200 lb. per sq. ft. Twenty-foot gage crane tracks are provided on each side of the pier to accommodate 50-ton drydock locomotive cranes.

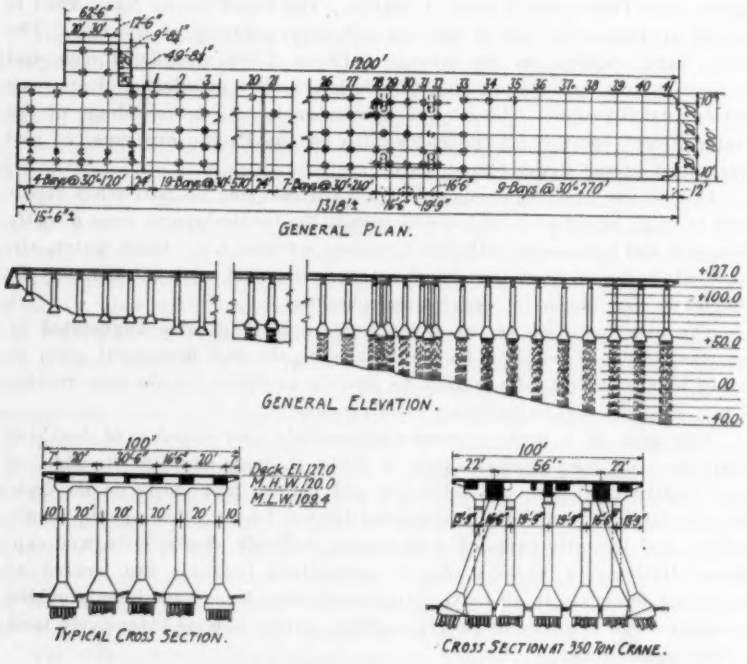


FIG. 1.—DETAILS OF REPAIR AND FITTING OUT PIER.

The pile bearing cylinders extend vertically from a point 4 ft. below the bottom line to the cross girder. The tops of the bearing piles are at the mud line and the pile extends into the mud a distance necessary to develop the desired carrying capacity (22½ to 25 tons). In the outboard section of the pier these bearing piles reached a length of 108 ft. All piles were driven by a floating driver with a steam hammer and 60-ft. built-up steel column follower.

The specifications required that all concrete work be placed in the dry and suggested the use of a precast shell or wood forms, either of which when sunk in place were to be unwatered by the pneumatic caisson

method and after excavation had been completed, filled in the dry with concrete. Precast concrete shells were required up to 60 ft. in length, 4 ft. 6 in. diameter shaft, 15 ft. 3 in. diameter bell, and weighed up to 60 tons. The casting and handling of these shells without injury was not an easy task. The most outstanding feature of this construction was the first extensive use of high alumina cement on a structure of such importance and magnitude.

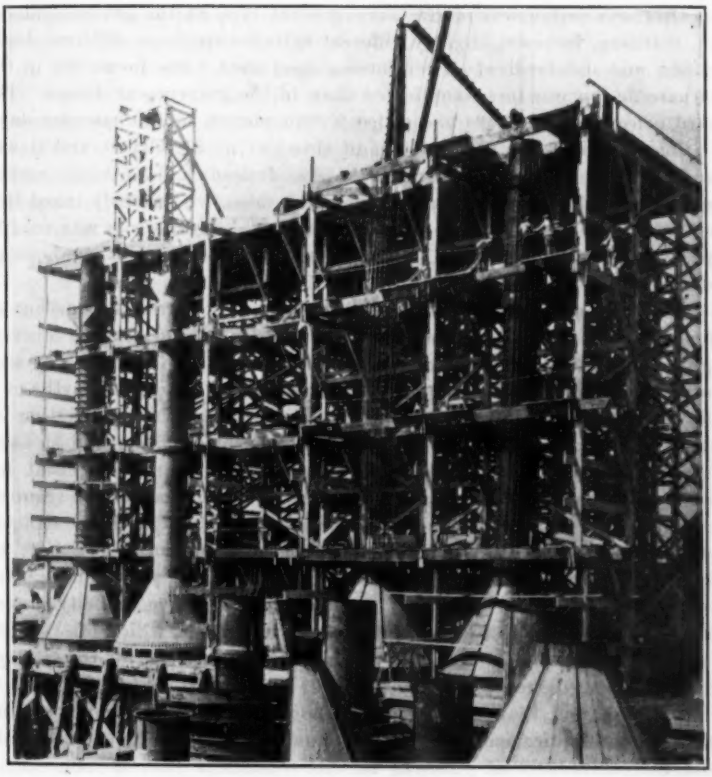


FIG. 2.—GENERAL VIEW OF CASTING PIER.

Before the adoption of the cylinder type of construction the yard had investigated the economics of a solid fill design, reinforced-concrete pile design, and combinations of both of these with the cylinder type. The cylinder design proved to be the most advantageous. Several designs for the crane foundation were investigated but the cylinder type with inclined brace cylinders to provide lateral stiffness proved to be the most economical.

The government prepared complete design and specifications for the

construction of this pier by contract, and invited proposals on this design at the same time giving bidders the opportunity of submitting an alternate design of equal strength and durability. This would permit a contractor to develop a design particularly suited to any special type of equipment he might possess or to utilize some construction method in which he was particularly experienced.

Award was made to Henry & McFee of Seattle, on an alternate design prepared by their chief engineer, W. F. Way, a member of this Institute. The alternate design was of the same general type as the government design, utilizing, however, slightly different cylinder spacings, different beam sections, and standardized on reinforcing steel sizes. The formwork in the alternate design was less complicated than in the government design. The award price was $1,167,394 and called for completion in 540 calendar days.

Congress fixed the limit of cost of this pier at $1,190,000, and it was found that to obtain a structure of the size desired, and we badly needed the size planned to accommodate four capital ships, we seriously taxed this limit. In searching for ways and means to cut down costs it was readily appreciated that any savings had to be made in construction methods as material costs were fixed beyond our control.

One item which prominently showed high costs was the casting of cylinder shells. Casting these shells of 1:1:2 portland cement concrete required that a casting yard be developed some distance from the pier site, crowded conditions precluded a casting yard nearer than three-fourths of a mile. The development of this casting yard meant the installation of tracks, gravel and sand handling equipment, cement handling and storage, mixing plant, reinforcing steel handling and storage, and equipment for handling sections of the finished shells. All this equipment was required in addition to similar equipment at the pier site, unless the contractor elected to cast all his cylinders before beginning work at the pier, an uneconomical procedure. Casting shells at the casting yard, unless very extensive and expensive temporary construction and large capacity cranes were provided, necessitated casting the shells in sections which later would be spliced and the splice then allowed to cure for a period of thirty days before the finished shell could be handled.

It is apparent that any procedure which would eliminate the casting yard with its duplication of plant and handling charges would materially assist in cost reductions. It was in this connection that the Bureau of Yards and Docks investigated the possibilities of using high alumina cement in casting the shells. If by its quick hardening properties it permitted the handling of shells 24 hours after casting had been completed, the necessity for long curing periods was eliminated, and if the shells could be taken away 24 hours after casting they could all be cast in the one spot and special temporary construction necessary for casting would be reduced to a minimum. As for handling, why not cast them adjacent to deep water, then the floating derrick which would place the shell in position in the pier could do all the handling. All this appeared reasonable and possible

and proved to be a cost saver despite the fact that the high alumina cement, manufactured domestically on the East Coast, would cost delivered on the job approximately three times as much as portland cement.

High alumina cement was consequently authorized for use in casting the cylinder shells. At the same time due to the higher strength of the alumina cement the shells were reduced in thickness from 6 in. to 5 in. Before actual work was undertaken, however, the contractor reverted to the 6-in. shell with no increase in contract price, it being believed that the material saving in a 5-in. shell over a 6-in. shell was offset by increased labor costs. Experience on the job proved this contention to be sound.

It will be recalled that high alumina cement was the only cement which successfully withstood the action of sulphate bearing waters in lining certain tunnels in France. It being recognized that the sulphates in sea water are the cause of most of the disintegration of concrete in sea water, high alumina cement should be well adapted for use in sea water. Specimens of alumina cement briquettes placed side by side with similar specimens of portland cement briquettes in concentrated magnesium sulphate solution showed little or no disintegration when the portland briquettes were completely disintegrated. The cylinders in this pier have been in place now for a little over a year and show no signs of disintegration. It is realized, however, that this time is far too short in which to draw final conclusion.

The comparatively meagre data available on alumina cement contained the results of many tests showing that alumina cement concrete bonded very well with older portland cement. No information was available, however, showing how well new portland cement concrete would bond with older alumina cement concrete. It will be noted that our construction contemplated filling the alumina shells with portland cores and to determine the effectiveness of this bond, sections of the cylinder shafts of one-quarter size were cast, using an alumina reinforced shell and portland core. The shell extended ¾ in. below the core on the bottom of the test specimen and the core projected above the shell ¾ in. on the top. (Table I). The specimen was placed in a standard compression machine and loaded to failure. It will be noted that the load was applied directly to the core only on the top, and the shell only on the bottom, the tendency being to push the core through the shell. The entire resistance offered, therefore, was in bond between the two concretes. Failure in each instance was by bursting of the shell at the top. The surface of contact was not roughened in any way—the surface left by the steel form of the shell not having been disturbed. The bond stress developed was such that if failure in bond was assumed to occur simultaneously with the bursting of the shell, the bond stress necessary to develop the full strength of the cylinder would be developed in 6 ft. of shaft. There was no question, therefore, of the effectiveness of the bond between new portland and older alumina concrete in these cylinders.

The original specifications were modified in accepting the contractors' bid to permit the use of gravel of "commercial grading with maximum size of 1½ in., uniformly graded, with not to exceed 15 per cent passing a ¼-in. screen. Sand to be commercial grading 100 per cent passing a No. 4 sieve and not more than 60 per cent nor less than 40 per cent passing a No. 30 sieve."

This type of aggregate was readily procurable locally and is practically the same as the city of Seattle standard. Maximum and minimum curves of this specification were plotted on forms and the sieve analysis of the various barges delivered plotted on these forms. The agreement with the specifications was immediately evident from the graphs.

The fineness modulus of the sand, on Prof. Abrams' scale, averaged 2.88 and of the gravel 7.25. Several experimental combinations were analyzed and the following mixes utilized on various parts of the work:

Nominal Mix	True Mix	Fineness Modulus
1 : 2 : 3½	1 : 4.28	5.66
1 : 2 : 4¼	1 : 4.76	5.85
1 : 1½ : 3	1 : 3.45	5.79

The specifications for concrete called for a fixed quantity of cement and coarse aggregate per cubic yard with a variable amount of sand. Strict application of this specification produced mixtures too lean in sand and as the result of actual tests the above mixes were adopted. No differentiation between the use of portland or alumina cement was made in analyzing the aggregates, it being presumed that what was suitable for one would be equally suitable for the other.

The specifications required a:

1 : var : 4 not less than 1.5 bbl. cement per cu. yd. for alumina shells except crane foundation.

1 : var : 3½ not less than 1.75 bbl. cement per cu. yd. for alumina shells in crane foundation.

1 : var : 4¼ for cores and deck concrete—2,000 lb. ultimate strength.

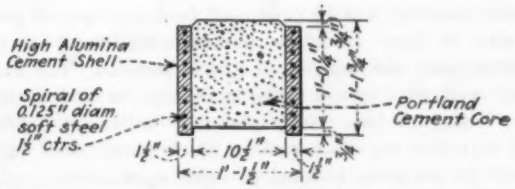
1 : var : 3 1/5 for concrete piles in crane-track approach.

In the crane foundation core concrete of 2,700 lb. ultimate strength was required and the mix was specified as 1 : 1½ : 3.

All high alumina cement was shipped direct from the factory to the job by water, requiring about one month's travel time. All high alumina cement was tested at the mill by the Bureau of Standards.

In preliminary experiments with high alumina cement standard 6 x 12-in. cylinders were cast of the following mixes:

1 : 1½ : 4, 1 : 1¾ : 3¾, and 1 : 2 : 3½; they were tested with slumps of 4 in. and 6 in. and compression values for 24 hr. ran from 4,980 lb. for the 1 : 1½ : 4 mix to 4,285 lb. for the 1 : 2 : 3½ mix:



CYL No.	SHELL ★ PROP.	WATER	ROUGH	POURED	CORE PROP.	WATER	ROUGH	POURED	REMARKS	TOTAL LOAD	LB. PER SQ. IN.	FAILURE	
1	1:1¾:3¾	7/12 Wt.	60%	5-29	1:2:4¼	⅔	1"	6-1	No allowance for bulking in sand	65760	760	Shell split	7 Days
2	1:1¾:3¾	⅔ "	40%	6-1	1:2:4	10/11	3"	6-2	Allowance for bulking in sand	68680	790	Shell split 4 places comp. failure top core	7 "
3	1:1¾:3¾	⅔ "	20%	6-2	1:2:4	9/11	5"	6-3	Sand and gravel wet	51110	590	Shell split 2 sides	7 "
4	1:1¾:3¾	⅔ "	25%	6-3	1:2:4	9/11	7"	6-4	Sand damp	75270	870	Shell split 1 place	28 "
5	1:1⅝:2⅝	½ "	5%	6-4	1:2:4	8/11	6"	6-5	3% Celite in core	40530	468	Shell split 1 place	7 "
6	1:1¾:3¾	⅔ "	15%	6-5	1:2:4	8/11	6"	6-6	Sand wet	85660	1000	Shell split 3 places	26 "
7	1:1¾:3¾	⅔ "	5%	6-6	1:2:4	9.5/11	5"	6-7	3% Celite in shell	68050	785	Shell split 3 places	25 "
8													

Note:- *Surface between steel and core showed signs of tearing on upper half only.*

★ *That part of surface which was not perfectly smooth due to gravel showing etc.*

TABLE I.—RESULTS OF TESTS.

Proportions	1 : 1½ : 4 Cu. Ft.	Wt., Lb.	1 : 1¾ : 3¾ Cu. Ft.	Wt., Lb.	1 : 2 : 3½ Cu. Ft.	Wt., Lb.
Cement	0.15	14.1	0.15	14.1	0.15	14.1
Sand	0.225	22.95	0.2625	26.77	0.30	30.6
Gravel	0.60	64.2	0.5625	60.19	0.525	56.17
Cu. Ft. Water	0.094	0.101	0.100	0.104	0.104	0.110
Gal. per Cu. Yd. ..	27.5	29.2	29.0	30.0	29.8	31.2
Slump, In.	4	6	4	6	4	8
Vol. Fresh Concrete, Cu. Ft.	0.692	0.698	0.695	0.701	0.705	0.712
Wt. per Cu. Ft. ...	155	154	155	154	153	151
Shrinkage on Setting, Approx..	0.01	0.012	0.01	0.012	0.01	0.013
Bbl. Cement to Cu. Yd., Fresh ..	1.48	1.45	1.46	1.44	1.43	1.42
Lb. per Sq. In. Compression						
24 Hr. (2 cyl.) ..	4,980	4,340	4,850	4,830	4,285	4,230
7 Day (1 cyl.) ...	5,735	4,850	5,610	5,140	4,670	5,150

The above test cylinders were made by placing concrete in steel forms and hammering the outside with a light hammer to jar concrete into place.

The 1: 1½: 4 mix was deficient in sand and numerous gravel pockets resulted. The tests of these cylinders showed considerable variation in strength, depending upon the amount of gravel pockets. The 1: 1¾: 3¾ mix worked very well with considerable vibrating—no gravel pockets resulting. The 1: 2: 3½ mix was easily placed in both consistencies. The aggregates used were in a dry state, and for above proportions and results, corrections should be made for bulking of moist aggregates.

The first cylinder shells were cast using a 1: 1¾: 3¾ mix with an allowance of one-sixth for bulking of the sand. This latter figure was obtained by actual measurements in the field. Water was proportioned to give a 6-in. slump and in the original batches an arbitrary allowance of 2½ gal. for each cubic yard of concrete was made for the water in the sand. Twenty-seven and one-half gallons of water per cubic yard would give approximately a 6-in. slump and a concrete of a consistency that handled well. The water was controlled by the mixer attendant and the quantity measured by a gage glass on the water barrel. The mixer man had difficulty in gaging the water where an extremely wet or dry batch of aggregate reached the mixer and it was observed that even a slight increase in the water content caused a noticeable increase in the consistency of the mix and the slump. This increase of water would cause the gravel to separate and be thrown on the top of the batch as the mixer was dumped and again when the concrete was poured into the forms. A change of 1 gal. of water per cu. yd. was sufficient to change the slump from 4 in. to 6 in., which indicated a rather well-defined critical point as to water content. It was realized that a 1: 1¾: 3¾ mixture was lean in sand and might cause difficulty in placing the concrete in the forms and despite energetic spading and continued hammering with air hammers on the outside of the steel forms the shells poured with this mix did not possess the uniform surface desired. It was decided to change the mix to a 1: 2: 3½ and better shells were produced as a result. When gravel pockets occurred in the shells they were chipped out to sound uniform concrete and replaced by a button patch of double the original shell thickness.

The outstanding features noted in connection with the use of alumina cement were the necessity of definitely controlling the water content, the necessity for well-graded and properly-proportioned aggregates, sufficient but not excessive time in the mixer, and effective tamping. It was also very evident that haphazard methods which are sometimes unfortunately used when inexperienced or uninformed persons handle portland cement concrete, cannot be utilized in handling high alumina concrete with relatively equal degrees of success. The special interest taken in the use of this new cement on the job caused it to be dubbed by the workmen, "high-hat" cement.

Contrary to a rather general belief, high alumina cement is not quick setting. It is quick hardening but not quick setting and no special effort need be made to hurry the placing of the mixed concrete into the forms. Care should be taken, however, to keep portland cement and alumina cement

separated and mixers used for both types of cement should be thoroughly cleaned with each change of mix.

All alumina cement used on this pier was shipped directly from the mill and used in a short time after arrival. Some alumina cement purchased for yard use from a local jobber and held in storage for approximately one year lost a large percentage of its quick-hardening properties. If the storage of high alumina cement for long periods before use is probable, it should be purchased in watertight and airtight containers.

Table II is a résumé of the test of the high alumina concrete used in cylinder shells.

TABLE II.—SUMMARY OF TEST CYLINDERS.

Cement	Mix	Age	Number of Cylinders	Average Slump, in.	Average	Maximum	Minimum	Location	Per Cent of 28-Day Strength
High Alumina	1:2:3½	24 hours	9	6¼	4,825	5,790	4,012	Shells	92
" "	1:2:3½	7 days	16	6½	4,642	5,350	3,118	"	88
" "	1:2:3½	28 days	23	6¼	5,256	7,080	3,980	"	100
" "	1:1.9:2½	7 days	38	6½	4,795	5,961	2,575	"	91
" "	1:1.9:2½	28 days	19	6½	5,257	8,005	3,095	"	100
" "	1:1¾:3¾	24 hours	3	6¼	4,352	4,893	3,754	"	74
" "	1:1¾:3¾	28 days	2	5¾	5,399	5,441	5,357	"	100

TABLE III.—SUMMARY OF TEST CYLINDERS.

Cement	Mix	Age	Number of Cylinders	Average Slump, in.	Average	Maximum	Minimum	Location	Per Cent of 28-Day Strength
Portland	1:2:4¼	7 days	33	2¾	1,741	3,433	1,130	Cores	54.5
"	1:2:4¼	28 days	29	2¼	3,196	4,285	1,652	"	100
"	1:2:4¼	60 days	22	1¾	4,101	4,996	2,890	"	129
"	1:1½:3	7 days	31	2	2,811	3,330	1,967	"	63.7
"	1:1½:3	28 days	31	2	4,421	5,198	2,260	"	100
"	1:1½:3	60 days	28	2	5,596	6,637	3,460	"	127
"	1:2:4¼	7 days	80	4	1,516	2,252	933	Deck	55.5
"	1:2:4¼	28 days	69	4	2,726	4,265	1,160	"	100
"	1:2:4¼	60 days	60	3¾	3,548	5,086	2,171	"	130

The advantage leading to the use of alumina cement was the fact that the cylinders could be handled 24 hours after casting was completed. For ease in pouring and handling, pouring in a vertical position was to be desired. A casting plant was designed consisting of a scaffold, erected on an abandoned timber pier adjacent to the new pier site. This scaffold was 53 ft. high, sufficient to pour the top of the maximum length cylinder. Adjacent to this scaffold a temporary casting pier was erected with deck at El. 122, 2 ft. above M. H. W. Scaffold and casting pier were of length sufficient to permit work on five cylinders simultaneously. On top of the scaffold with boom reach over the casting pier a stiff-leg derrick was erected with the operating engine on old pier deck. This derrick was used for handling cylinder forms, setting reinforcing steel, etc.

It was intended to lift the cast shells from the deck of the casting pier by the large floating derrick with a choker around the shaft of the shell, the bell of the shell to abut the bow of the derrick barge to prevent swinging. This, as well as the limitations of the derrick, prevented a vertical lift from the casting pier sufficient to clear the inside bell form. At each cylinder position on the casting pier a clear opening in the deck, and below it, sufficient to permit the top of inside bell form to drop below the deck of the pier was provided. This inside bell form was made up of two halves, each a timber frame covered with lath and metal and supported above the pier deck by dogs, from the pier, engaging recesses in the bell form. The two halves were wedged into final position. In removing the

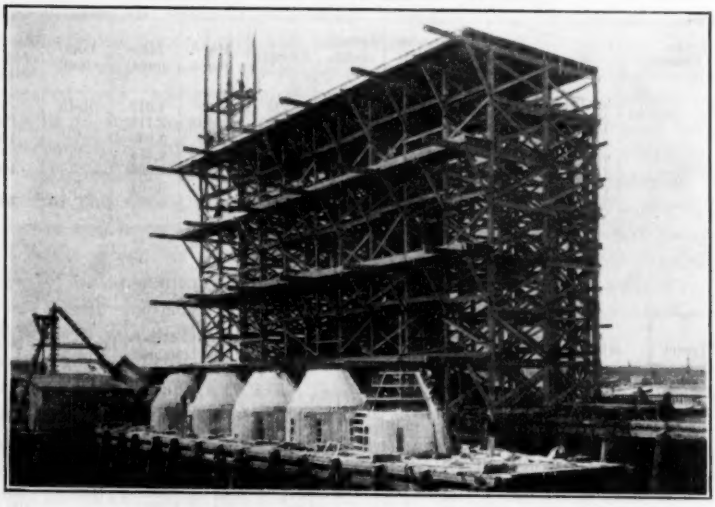


FIG. 3.—VIEW SHOWING CONSTRUCTION OF INSIDE BELL FORMS.

bell forms the wedges were struck, dogs released and the scaffold derrick engine, by block and tackle, pulled the form below the pier deck.

Variations in inside bottom bell diameter and the height of the bell were obtained by engaging the dogs into recesses at varying heights along the bevelled lower section of the inside bell form. Corresponding changes in the outside steel bell form were made by removing sections of varying lengths from the vertical bottom section of the outer form.

The outside conical and skirt section of the bell form was of steel in two halves. The shaft forms were semi-circular in section and of a height equal to the distance between intermediate pouring platforms of the scaffold, with smaller sections as necessary to give the desired final length of cylinder. All outside forms were secured by McFee bolts—a through rectangular bolt with a wedge shaped cotter pin. The inside shaft form

was originally in sections of the same lengths as the outer form and had a full length center king pin to which were attached radial rods, which when the form was lifted by the king pin were supposed to automatically collapse the form along an open vertical joint. In practice, however, the concrete held the forms too securely and the pull on the king pin only broke the radial rods. These forms were superseded by a steel form, one piece for full length of shaft, open along a vertical element which open joint was controlled by turnbuckles in chord rods attached to each side of the joint. To remove forms these turnbuckles were drawn up by hand, thus

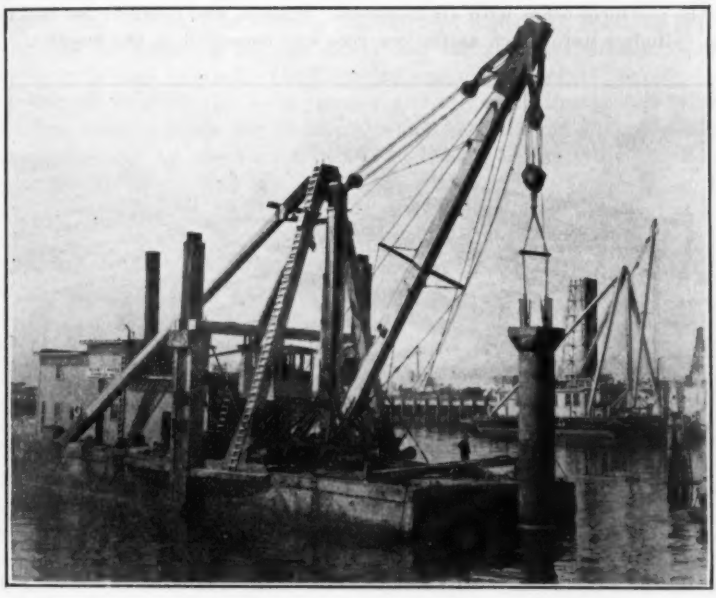


FIG. 4.—DERRICK HANDLING CYLINDER INTO PLACE.

breaking the bond with the concrete and the full length form was lifted out by the derrick at the top of the scaffold. This lift taxed the lifting height of the derrick to the limit.

Outside bell forms and inside shaft forms were stored by the derrick on the old pier deck while the outside bell forms were stored on top of the scaffold. A special top form was used to cast a rectangular corbel on top of the cylinder shafts to provide support for the deck forms.

In casting the shells the inside bell form was first set in position, the bell reinforcement wired into place, and the outside bell form placed. The spiral shaft reinforcement (received on the job already spiralled) had been previously spaced on two of the main reinforcement rods of the shaft and

the resultant frame was set on top of the bell forms and temporarily secured to the scaffold. The remaining vertical reinforcement was then placed and the inside shaft form set. After the bell had been concreted the first section of the outside shaft form was placed, this section poured and the process repeated to the top of the shell.

Concrete was mixed in a floating plant and by derrick deposited in the hopper of a concrete elevator located at the end of the scaffold. The concrete was elevated to the intermediate level desired and wheeled in buggies to the forms. Concrete was placed at opposite sides of the shaft, spaded by men on the pouring platform and the forms hammered by men on the platform below with air hammers. Slumps, and concrete for making test cylinders were taken as the concrete was deposited in the buggies.

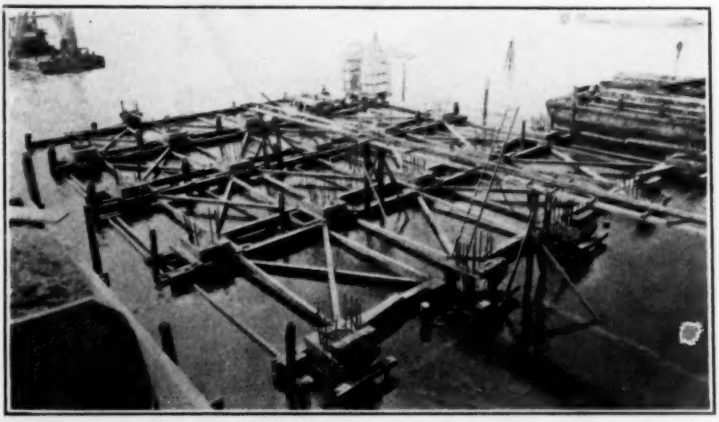


FIG. 5.—CYLINDER BRACING SYSTEM.

As the work progressed the men became more familiar with the handling of the concrete and forms and we cast a maximum of three cylinders per day. Exposed surfaces were kept wetted and forms were removed in from 16 to 20 hours.

Shells were lifted from the casting pier only at high tide and the cable choker wrapped around the shaft was entirely satisfactory for this purpose. As the shell lifted from the pier deck the bottom immediately kicked out from pier till it came in contact with the bow of the derrick scow. This movement was inevitable due to the eccentric application of the lifting force. With the cylinder thus supported the derrick was backed away from the casting pier and the cylinder lowered until it rested on the bottom. Further handling and lifting was by a bridle shackled to two inverted U-bolts cast into the top of the cylinder shells. The shells were then removed either to be placed immediately into position in the pier or to a storage site.

One hundred and nineteen of the 234 cylinders in the pier were soil bearing, the remainder were supported on timber pile clusters. Pile driving was carried on well in advance of the other construction. Some dredging was performed to remove the excess soft material at the site of the soil-bearing cylinders. Construction work on the pier proper was started from a temporary anchor pier erected across the full width of the proposed pier at the end of the existing timber pier. The first bent of cylinders was set in place by the floating derrick and braced to this anchor pier. The cylinder when carried by the derrick was supported truly vertical by the bridle aforementioned and was carried approximately 75 per cent submerged. The derrick was securely anchored to temporary dolphins previously used for handling the pile driver and the cylinders were set quite accurately in final location. Until bracing was set and bolted the cylinders were secured by cable ties to the anchor pier. A fabricated timber framework was used to brace the cylinders, which consisted of a chord timber (transverse with the pier) on each side of the top of the cylinders, cleats on the inner side of these chord timbers formed a guide for securing the cylinder, and diagonal cross-braces pin connected to these chords and extended to the next forward bent where the chord member was temporarily supported on piling. The next bent of cylinders was then set in place and the process repeated.

When cylinders were thus fixed in location a concrete spider was set on the top of the cylinder on which were piled weights composed of bundles of rail steel, or concrete blocks, to resist the upward thrust of the air pressure, and assist in sinking the soil-bearing cylinders. A steel air lock of the same diameter as the inside of the cylinder shell was placed on top of the cylinder, secured to it by tie rods to the U-bolts and an air-tight joint made between top of cylinder and bottom of lock by expanding a one-piece rubber skirt, attached to and projecting below the lock, against the inside cylinder wall. Compressed air, provided from the yard mains, was applied to the cylinder and all water was expelled from the inside of the shell by blowing out under the cutting edge. This water also carried with it large quantities of the softer materials from the bottom. When all water was expelled sand hogs entered and rigged their lights, rope ladder and water jet line. In the soil-bearing cylinders excavation was begun by jetting the material and blowing it out under the cutting edge. As the material became harder the excavated material was removed by buckets hoisted through the air lock by means of a small headframe and compressed air engine rigged on top of the cylinder. In some cases dynamite was successfully used in loosening the hard material.

When excavation was completed and the foundation inspected and passed the cylinder was immediately concreted. A concrete lock was developed on the job which permitted the placing of a full batch of concrete in the cylinder in one locking operation. This lock was cylindrical in shape with a conical bottom tapering to make a flanged connection with the flange of the top door of the air lock, no bottom door was provided in the con-

crete lock. The top door of this lock as well as the two doors of the air lock was a counterbalanced door operated by a lever on the outside of the lock—these doors were round in section, perfectly flat and when closed were held against a gasket frame set in the lock by the counterbalance and the air pressure in the lock. In concreting, the bottom door of the air, or manlock, was constantly open, the top door of the air lock, which when concreting was also the bottom door of the concrete lock, was closed, the cylinder being under pressure. The concrete lock was filled, top door closed and pressure in lock and cylinder equalized when the top door of the air lock was opened and concrete dropped by gravity to the bottom of the cylinder.

FIG. 6.—EXCAVATING CYLINDERS SHOWING AIR LOCK, WEIGHTS AND TRUSSES FOR SUPPORTING DECK FORMS.

This method worked very satisfactorily and was a decided improvement over a previous method where concrete in quantities of one-third of a batch was forced into the cylinder by high pressure air and where water used to clean a gasket after each operation found its way into the core concrete. The concrete was delivered to the cylinder by the stiff-leg derrick on the floating mixer plant. The time for locking through about equaled the mixing time so the filling of cylinders was not, under the new method, a tediously slow job. In concreting these cylinders one thing which must be absolutely controlled to insure results is uniformity of the air pressure. We realized this and saw to it that nothing interfered with our air supply —at first, however, we did not get satisfactory results—the concrete placed was watered and had to be removed. There was no apparent cause for this watering. We decided then to follow the filling of a cylinder by

inspecting the concrete after every two or three batches to see when the water entered. The inspector would enter the cylinder after every two or three batches and would bring out a sample of the concrete perfectly o.k. We thought after several inspections that in this particular cylinder we would get no water when the next time the inspector entered the cylinder he had trouble with the lock door—the gage tender noted a sudden deflection downward of the needle which almost immediately returned however to the operating pressure. When the inspector arrived inside there was the water. The inspector did not realize that the slight trouble with the door could have caused a drop in air pressure and it would not have been noticed had not the gage man been on the job. The positive and continuous

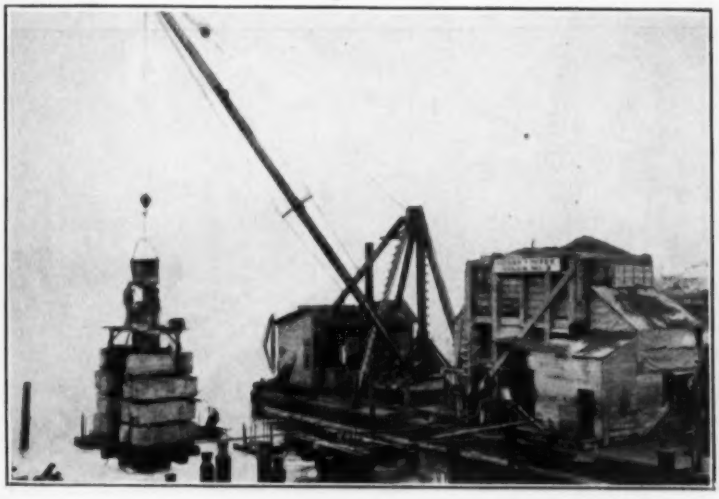


FIG. 7.—FILLING CORE OF CYLINDER THROUGH CONCRETE LOCK.

supply of air during concreting is of the utmost importance. As the concrete filled the cylinder we reduced the air pressure gradually but did not relieve the pressure entirely until the concrete had taken its initial set. Another factor which must be considered in concreting shells which require such a large volume of concrete for filling is the tendency of the shell to lift due to the upward thrust of the wet concrete—that upward thrust is there, we know—we saw what it could do when not completely resisted.

We used slumps averaging about 2 in. on this core concrete without any difficulty in handling, and on the richer cores of the crane foundation cylinders we at times had as low as 1-in. slumps.

It will be recalled that these cylinders were in some cases 60 ft. long—the question has been raised that we could not drop concrete through this

height without serious segregation of the ingredients. Due to the uplift of the concrete core displacing a cylinder we had to remove the core concrete down to the bottom edge of the cylinder, which gave us a wonderful opportunity to inspect the quality of this core concrete. Throughout the entire length of the shaft and bell there was not a sign of any segregation whatever, the concrete was in excellent condition. The presence of minute air bubbles was apparent, they were very small and scattered and were formed by the concrete entrapping the compressed air.

The placing and filling of the pile bearing cylinders differed in no respect from the soil bearers excepting that the work of excavation consisted only in removing the mud from around the tops of the piles and clearing any interference of cylinders and piles.

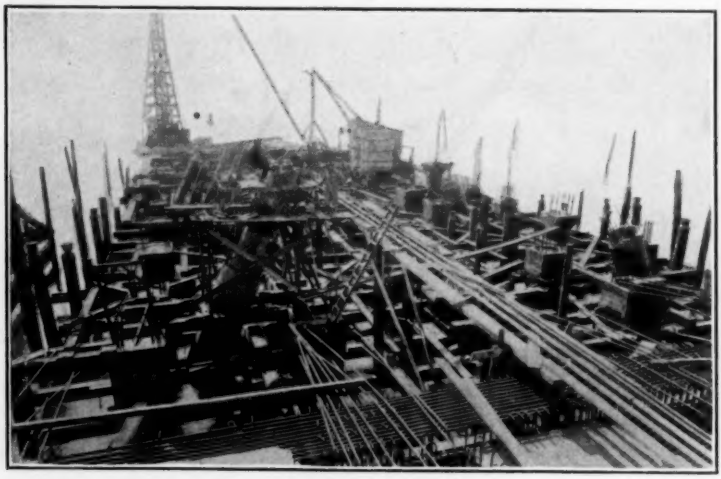


FIG. 8.—CONCRETING INCLINED CYLINDER, SHOWING INCLINED PILES BEFORE CUT OFF.

The pile driving on this job is of interest due to the accuracy with which the piles were driven and the length of piling handled. Clusters completely circumscribed by a 14 ft. 3-in. diameter circle were driven, containing 36 piles with an average diameter at butt of 14½ in.

At the crane foundation the inclined brace cylinders were supported on inclined piles. These piles were driven by a drop hammer in leads hanging from a floating derrick. They were driven as long piles and later cut off by diver at the mud line. The inclined cylinders were cast in the same manner as the vertical cylinders without the corbel top. The bevel required in the bell bottom was obtained by placing a bulkhead in proper position in the bell form.

The brace cylinders were held in position at the bottom by cables secured to a cable loop, one end of which passed under the edge of the adjacent cylinder and was secured around the piles and concreted in place when the cylinder was filled. These cable ties were secured to a cable girdle around the top of the bell of the inclined cylinder. The top of the inclined cylinder was secured by cable ties to adjacent cylinders. These cylinders were concreted in the same way as the vertical cylinders.

The deck forms were completely designed and fabricated before the cylinders were placed. They were standardized in panels for sides and bottoms, were easily handled and set in place with a minimum amount of carpentry work.

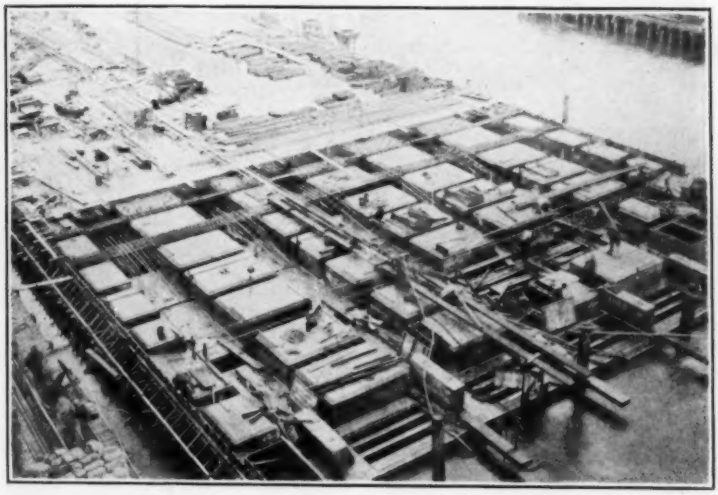


FIG. 9.—FORMS IN PLACE FOR CONCRETING DECK.

The entire deck form system was supported on inverted queen post trusses resting on split blocks placed on the corbels at top of cylinders and spanning to the corresponding cylinder in the adjacent bent. On the upper chord of these trusses 8 x 18-in. timbers were placed transversely for the full width of the pier. These 8 x 18-in. timbers were spaced 18 in. on centers and thus formed a level floor system over the entire pier area. The elevation of the floor system was adjusted by varying the thickness of split blocks so that the bottom forms of all beams and girders were placed directly on top of the 8 x 18-in. timbers. The side panels and deck slab panels were supported and braced by studding from the 8 x 18-in. timbers. The cross-girder forms were hung from the 8 x 18-in. timbers by steel rods and cleats.

In removing forms the split blocks under the ends of the trusses

were broken out, thus dropping the entire floor system and the 8 x 8-in. timbers removed. The removal of trusses and panel forms presented no unusual difficulties. The use of these forms of several standard fabricated parts greatly facilitated and expedited the field work.

The main reinforcing steel was composed almost entirely of 1½-in. deformed bars, rolled in Seattle to required lengths. All handling, bending and placing of steel was performed on the work by subcontract. The use

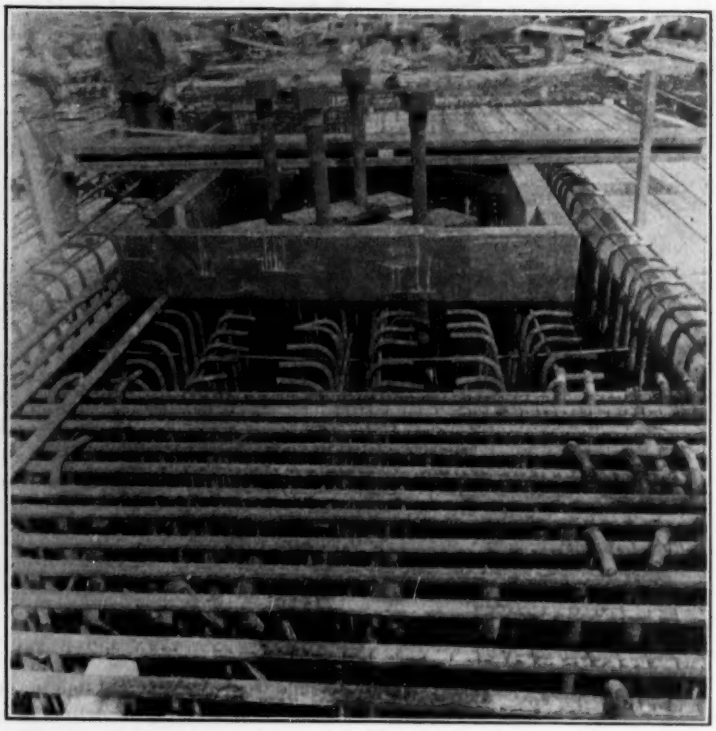


FIG. 10.—STEEL IN GIRDER ON CRANE LEG, SHOWING CRANE ANCHOR BOLTS.

of the long 1½-in. rods (they reached a maximum length of 62½ ft.), while at times cumbersome, greatly reduced the amount of handling required and at the same time permitted wider spacing of rods. All beams and girders had a net minimum cover of 2 in. and deck slabs 1½ in. The rod spacers were wedge-shaped cast concrete blocks with grooves for accommodating the 1½-in. rods. When rods and stirrups were wired in final position they formed a steel skeleton of considerable magnitude and stiffness.

The girder supporting each leg of the 350-ton crane was 9 ft. square in cross-section and contained as main reinforcement sixty-six 1½-in. rods. After approximately two-thirds of the steel had been placed it looked as though we could never find room for the remainder but we did, and also managed to place the four 3½ diameter anchor bolts, whose lower end framed into a built-up grillage, in the center of this girder.

Deck concrete including cross-girders, girders and beams was poured in full 30-ft. bays, approximately 330 cu. yd. of concrete per hour. This was readily handled by two floating mixers, one operating on each side of the pier with mixers of 1 and 1½ cu. yd. capacity. Concrete was delivered in bottom dump buckets directly to the forms. Each mixer had its separate gang of spaders. The cross-girders were first poured to the level of

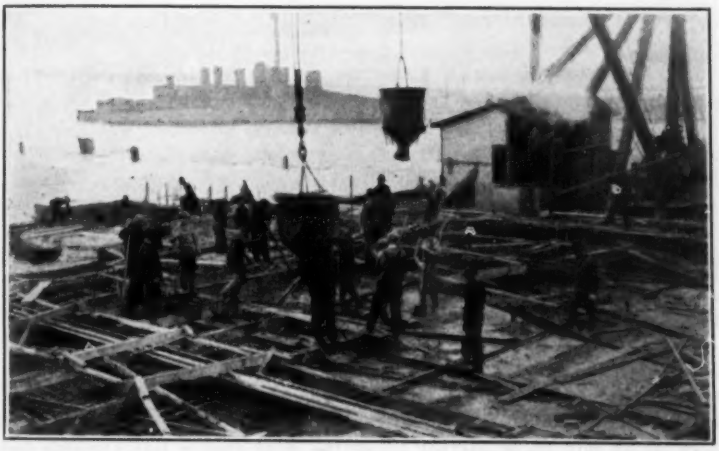


FIG. 11.—POURING DECK CONCRETE.

the longitudinal girders and the concrete then brought up level over the entire area. Slumps varied from 3½ in. to 6 in. Standard 6 x 12-in. cylinders were made from concrete removed from the forms after spading had been completed.

Anchor bolts for track plates, bollards, cleats, bitts and drains were set before deck concrete was placed. Electric ducts were concreted in place. That portion of manholes below the elevation of the 8 x 18-in. floor system was poured after deck forms had been removed.

The concrete structure is protected by a creosoted-timber fender pile system. At the corners blows received by a fender pile cluster are transmitted to the pier structure through a system of large spiral springs and pin connected oak chocks and wales. The entire pier deck is to be covered with a 1-in. bituminous carpet.

The 20-ft. dry dock crane track approach to the pier is supported on precast concrete piles and reinforced girders. When the original length of precast piles was too short to provide necessary resistance they were lengthened in place by building up with high alumina concrete and driving was successfully continued in 24 to 48 hr.

Work on this pier was begun on April 28, 1925, and entirely completed with the exception of the electric cables, piping, and bituminous carpet, on Sept. 7, 1926. The work progressed uniformly throughout the life of the contract and we were fortunate in experiencing no delays due to inclement weather. The controlling feature of the progress of the work

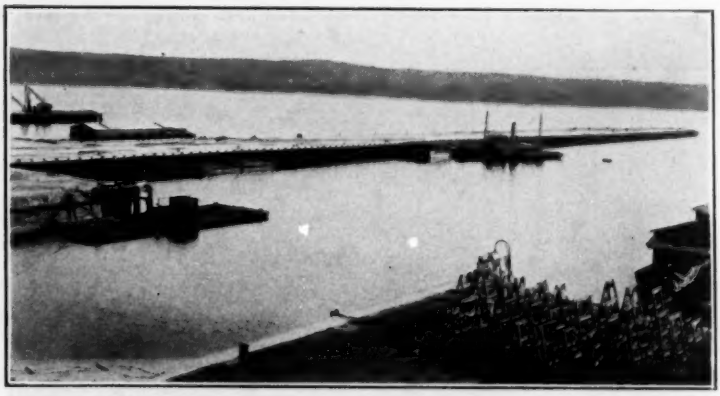


FIG. 12.—VIEW OF THE COMPLETED PIER.

was the excavation of the cylinders, particularly those soil-bearing cylinders which had to be carried below the final dredge line through hard material.

The final cost of the work was $1,186,250.42, or a cost per square foot of pier, including crane track, approach, and offset of $8.80.

This work was carried on under the direction of the Bureau of Yards and Docks, Navy Department, Rear-Admiral L. E. Gregory (C. E. C.), U. S. N., chief of bureau; Commander G. A. Duncan (C. E. C.), U. S. N., project manager. Capt. E. R. Gayler (C. E. C.), U. S. N., made the original designs and studies. The construction work was carried on under the supervision of Capt. W. H. Allen (C. E. C.), U. S. N., with the writer in charge in the field, assisted by E. C. Jack.

W. F. Way was chief engineer for the contractor and P. J. MacKenzie, superintendent.

Discussion—Construction of Reinforced-Concrete Pier.

W. F. Way* (*By Letter*).—The paper just read very adequately describes the methods used in the construction of this pier. There were a few fundamental principles employed in this design to facilitate construction that might be of sufficient importance to warrant mention. Mr. Way.

The first of these is the use of concrete foundation cylinders in which the l/d ratio is not greater than 15, in place of the standard reinforced-concrete pile. In general, but not in this case, however, reinforced-concrete piles have a less first cost than do cylinders such as used in this design. Usually the difference is small and may soon be cancelled by the maintenance charge. As a matter of comparison, a 4 ft. 6-in. diameter cylinder, similar to those used, has the load carrying capacity of eighteen 18-in. square reinforced-concrete piles of 40-ton live-load capacity.

The important point I wish to make is that the surface area of concrete exposed between the tides is 6.8 times more for the pile design than for the cylinder design, and certainly this is a factor that demands proper consideration because deterioration of sea water structures is pretty well confined to the exposed surface and any design which reduces this area 5 to 7 times should certainly be allowed a higher first cost, if necessary. It is also believed that the cylinder concrete when in place contains fewer defects than does that in the piles.

Another point of advantage possessed by the cylinders is that the outer shell may be precast in a better grade of concrete than that used to fill the core. This is also a logical design as the two concretes are subjected to entirely different conditions. In our particular case it was possible to use lumnite cement concrete for all exposed surfaces. The cost would have been prohibitive to use the same grade of concrete in piling, as it would have required 6 times as much lumnite cement, or, an additional 35,000 bbl.

It has been asserted that concrete piles are more easily repaired than are the cylinders, as a new pile may easily be driven adjacent to old pile simply by cutting a hole in the deck and later patching it when the connection is made between the new pile and deck. The question of comparing the relative cost of this class of repair work is indeed a difficult one; should the deck of the structure be covered with a building, then the problem of replacing piling is much more serious.

As a general statement, I do not believe that the repair work on cylinder is any more expensive than on piles. We have on two different occasions contracted to repair cylinders supporting a concrete deck and

* Chief Engineer for Henry & McFee, engineers and contractors.

found that the work was not so difficult. On one job 22 cylinders were successfully repaired by completely surrounding the old cylinder with a 6-in. thickness of reinforced lumnite concrete.

The second point for comment is concerning the design of the deck, and in this, attention is called to the article appearing in the *Engineering News-Record* March 3, 1927, in which typical cross-sections of various studies are compared. It will be noted that the design selected is made up of beams of very large cross-section having more than ordinary width. In comparison with a conventional design, the formwork was greatly reduced which is an economical point in the first cost; but exceedingly more important from an engineering standpoint is the 35 per cent reduction in the surface area.

Again sea-water deterioration varies as the amount of exposed area, and this variation is not necessarily a straight line as the concrete placed in the large sections should be better concrete than that placed in narrow deep members over-crowded with reinforcing steel. The plan selected approaches a mass design; no member is so small but that a workman could enter same after all the steel was placed, and this was often necessary in order to adjust steel and properly tamp the concrete. Such a criterion might well be employed to advantage for all outside structures. In general the reinforcement was 1½-in. round bars, on 4-in. centers, and placed in one layer. Where two layers of bottom steel were used, the concrete was made richer and workmen entered the forms in order properly to spade the concrete around the bars.

A great deal has been published lately in the engineering press about disintegration, corrosion and deterioration of reinforced-concrete structures. Naturally such discussion should result in better concrete work. Along this line the designer certainly has his part to perform. Generally speaking, I believe that most concrete members are far too small and so crowded with steel that a good job of concrete is impossible, and especially so when a *lean*, dry concrete is being used; lack of sufficient cement to make a workable mix is the cause of considerable poor construction.

Design of Concrete Buildings for Wind Stresses.

By Albert Smith.*

I wish to discuss three points in concrete building design in relation to wind stress:

1. What dependence, if any, should be placed on walls and partitions.
2. If no dependence is placed on walls and partitions, what is the ultimate strength of reinforced-concrete buildings against wind?
3. What devices can be used to increase the strength of such buildings against wind?

In regard to partitions and walls, I wish to advance the view that no dependence should be placed upon them.

In regard to the ultimate strength of the frame, I shall show that while, in some cases a ratio of 2 to 1 for height to width without special reinforcing gives a passably safe structure, that in most cases it does not and that the height-width ratio is an indication but not a guide.

In regard to partitions, it is apparent that a large number of partitions running parallel to the wind will have much stiffness and considerable strength. If these partitions were of reinforced concrete and poured with the floor, they would transmit wind shear directly from floor to floor and we should have no wind problem. Instead of this, 3 and 4-in. tiles plastered are carried up to the ceiling lath and only those which bear against beams and columns can develop resistance to wind deflection. Those that do so, bear on a small area.

In regard to walls, while the bond of the wall below to the spandrel above is slight, the bearing against columns at either end of a wall panel is 8 in. wide, and the material is better. Most walls, however, are cut by windows, and the ultimate horizontal shear strength of the average pier between windows is not great.

The horizontal wind shear in a 14-story building 60 x 140 ft. would be, in the first story, at 20 lb., about 400,000 lb. Instead of two walls 60 ft. long and 8 ft. high, we have about 40 ft. in isolated piers between windows which act as cantilever beams 6 ft. high.

It is, I think, evident that the combined effect of partitions and walls is entirely inadequate to resist the wind in buildings of any great height. There is a great temptation, however, to say that even though they cannot resist all the wind, they can surely resist some portion of it, as say, one-fourth or one-third. This should not be done.

The deflection, which the walls and partitions will endure without falling is very much less than that which the reinforced-concrete frame

* Smith & Brown, Consulting Engineers, Chicago.

will stand. Long before the maximum, the catastrophic wind pressure for which we design has been reached, a series of minor crushings and shearings will have made the wall and partition resistance negligible. A solid wall laid up in tempered mortar and wedged at the spandrel beams, will undoubtedly offer great resistance. Unless it is so laid the amount of resistance assigned to it should be small. If then, the strength of the walls and partitions cannot be relied on, the reinforced-concrete frame must be capable of taking all the wind load.

In high buildings, the columns are so thick and the beams so deep that it seems as if no wind could blow down a frame so heavy and so stiff. Just how strong is this frame? (See Fig. 2.)

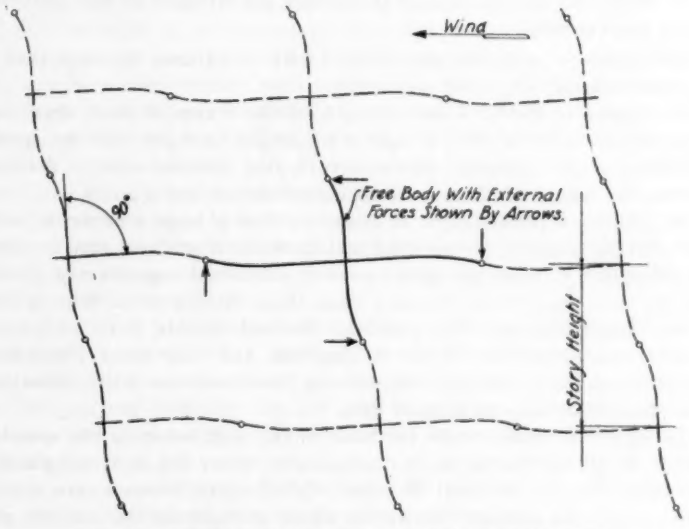


FIG. 1.—BENDING OF COLUMNS AND BEAMS FROM WIND.

To determine roughly their ultimate strength, I have assumed a series of square panels of varying dimensions, designed floor beams parallel to the wind, and computed the amount of wind stress which would cause them to fail.

In computing the ultimate strength of these beams, I have taken the ultimate strength of concrete in compression as 2,000 lb. and in bond as 400, in tension as 250 lb. I have taken the ultimate stress in the steel as 40,000, and have assumed that at the face of the column the bottom flange of a beam will have a compressive fibre stress of 500 lb. on the end where the wind gives tension, and 800 where it gives compression.

The dead- and live- and the wind-load stresses are here illustrated.

For the 13 x 24 beam, in panels 20 ft. x 20 ft., the ultimate resisting moment against wind as point A (Fig. 2) is made up of two parts (see also Fig. 3):

Resisting moment of concrete of 13 x 24 beam with 15 x 2½ flanges each side	98,000 ft.-lb.
Resisting moment of two 1-in. sq. embedded at least 9 in. in the column	24,000 ft.-lb.
	122,000 ft.-lb.

A 20-ft. strip of wall 1 ft. high receives 400 lb. of load. The horizontal shear in each column above and below the beam, in a one-bay building, is

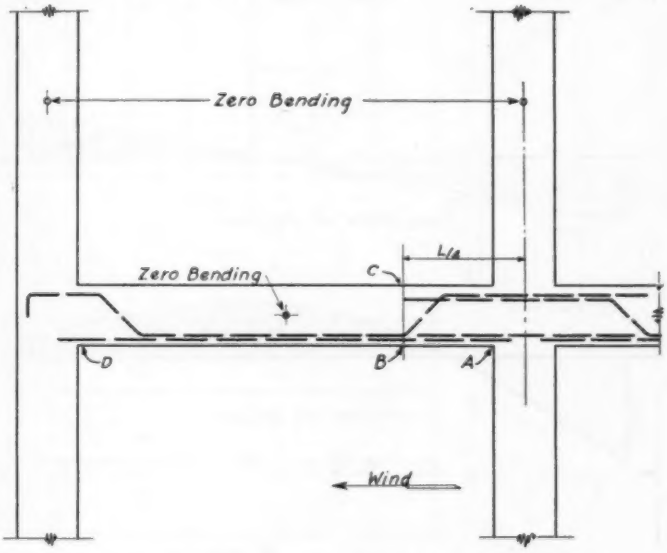


FIG. 2.—DIAGRAM OF CONVENTIONAL REINFORCING.

200 lb. The moment in the beam is, therefore, for 10-ft. story heights, 400 lb. x 4 ft. = 1,600 ft.-lb.

In a one-bay building, where the wind exposure above is 75 ft. the girders parallel to the wind will be at the point of failure. A two-bay building will fail under exposure of 150 ft. above, and a three-bay building will fail under 225 ft. of exposure. For shorter and shallower beams, or for narrower bays with shallower beams, the height-width ratios at failure are about the same.

Where for such spans beams in square panels have been made less in depth than one-tenth the span, the height-width ratio at failure becomes smaller. My calculations indicate that with a height-width ratio of two

we have, for such beams as are here examined, a safety factor of 1.87. Note that these calculations are made from the point of view of explaining why many slender buildings with no special wind bracing have successfully endured severe wind exposure.

Thee severe wind may come before the concrete has attained a strength of 2,000 lb. per sq. in. The concrete at the end of the beams is apt to be honeycombed. The bottom rods may have less than 9 in. of penetration into the column, and after all we have no assurance that while we design

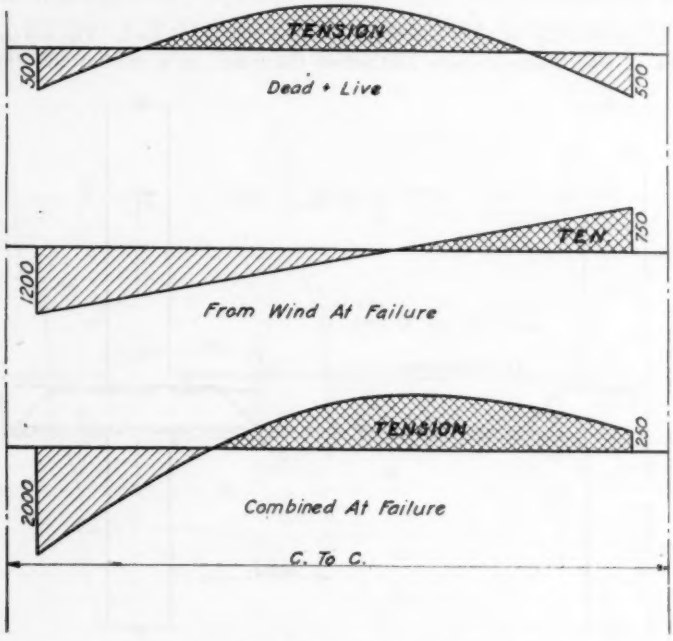


FIG. 3.—FIBER STRESS IN BOTTOM FLANGE.

for 20 lb. there may not occur a wind that will give 25 lb. per sq ft. For these reasons I prefer a safety factor of 2.5.

At point B (Fig. 2) the ultimate resisting moment is as follows:

As a concrete beam, with no dead and live fibre stress, C-250 lb. tension	33,000 ft.-lb.
Two bottom rods, 1 in. sq.	10,000 ft.-lb.
	43,000 ft.-lb.

or after the concrete has failed in tension

$$2.0 \times 40{,}000 \text{ lb.} \times 1.6 \text{ ft.} = 128{,}000 \text{ ft.-lb.}$$

which corresponds to a wind moment at edge of column of $2 \times 128 = 256{,}000$.

For compression at B and for tension and compression at C (Fig. 2), the ultimate resisting moment is far greater than can be developed in tension at A.

The simplest method of developing additional strength at A is by putting two additional rods through the column and in the bottom flange of the beam.

The ultimate resisting moment due to such rods will depend upon their size and length. Two 1 in. sq. as above will produce 256,000 ft.-lb. which corresponds to a height of 160 ft.

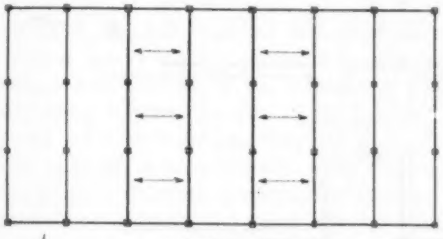


FIGURE 4

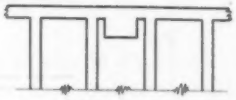


FIGURE 5

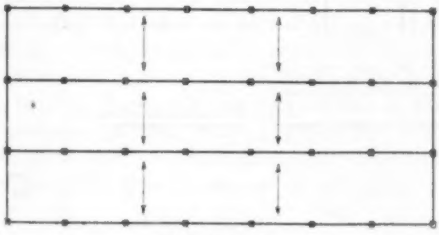


FIGURE 6

FIGS. 4, 5, 6.—WIND ACTION ON FRAMING ELEMENTS.

The building shown in Fig. 4 with 13 x 24-in. beams parallel to the major wind load and with wind rods as above would have a factor of safety of 2.5 in second floor beams if carried to a height of twenty 10-ft. stories. In addition to the bottom straight rods across intermediate columns, it is necessary to place bent wind rods in both top and bottom flanges of beams at corner columns.

Where the beams run along the longer dimension, we have two framing elements resisting the wind against the broad side (see Fig. 6).

The floor of a reinforced-concrete building may be regarded as a horizontal girder of great stiffness. All columns have their tops displaced the same amount (see Fig. 1). The flexible beam will not, therefore, develop

as much fibre stress as the stiff one. The joists and spandrel beams which must work together cannot both develop their ultimate strength.

It will take sixteen 5 x 12½-in. joists and thirty 5 x 10½-in. joists to offer as much resistance as the 8 x 24-in. spandrel beam. Assuming that four 5 x 12½-in. joists may be counted as effective at each column (see Fig. 5), we have a total ultimate resistance in this 60 x 140-ft. building of

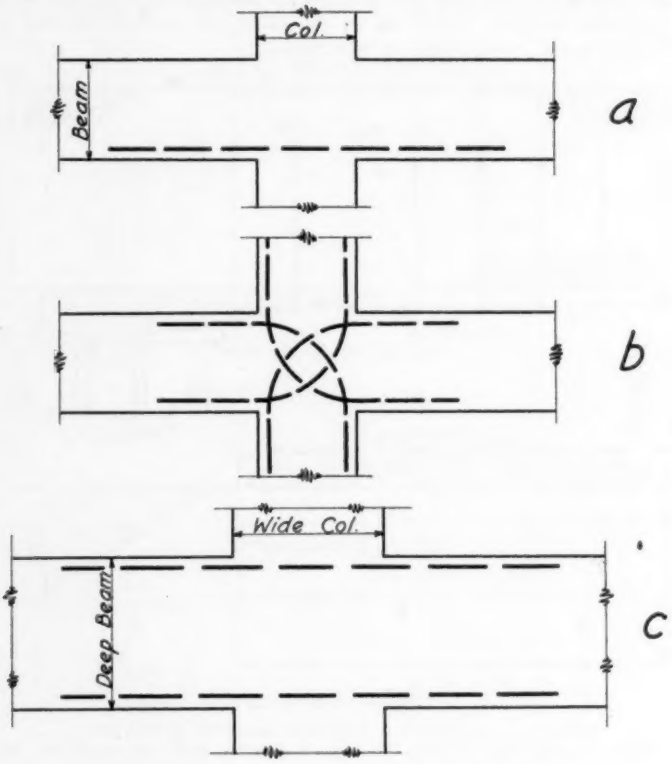


FIG. 7—A, B, C.—REINFORCING AGAINST WIND.

245,000 ft.-lb. at each end of each of the three bays. This is equivalent to a height of 66 ft. In such a case, special wind bracing methods must be used.

In buildings with wings, we often have beams of different depths and spans acting together. The product of the I/L of beam and column gives a coefficient. Add all coefficients together, divide into the total shear and multiply the result by each coefficient to give the shear resisted by each column. Where the building is unsymmetrical in its framing, the weaker half should be counted as resisting one-half the total wind.

Where the normal strength of beams and column or of beams only is insufficient the four methods, (a), (b), (c) and (d), given below, have been used to reinforce the structure against wind.

(a) Reinforcing all bottom flanges, and top flanges at corners only. Limited by bond in concrete of columns (see Fig. 7a).

(b) Reinforcing both top and bottom flanges of beams, and also reinforcing the columns (see Fig. 7b).

(c) Increasing the bond in the rods through the column by increase of dimension and increasing the strength of both beams and columns (see Fig. 7c).

(d) If, in the end walls we use 8 in. of concrete instead of 8 in. of backing brick, and reinforce the corners of all openings in such walls, we shall have a portal bracing adequate for any ordinary case.

I have said nothing of the strength of columns. This is because in ordinary story heights, the arm of the horizontal shears in the column is small, and for high buildings, fourteen to twenty stories, the columns are likely to be 24 in. to 32 in. square. Fifty per cent of the fibre stress in such columns gives a resisting moment that is large enough in ordinary cases.

First-story columns and second-story beams are likely to have exceptional stresses due to the fixity of some columns at the first floor, and due to extra height of the first story.

In looking over my figures, I do not find it wonderful that such concrete buildings as the Flamingo Hotel resisted the Florida hurricane. The framing plans of this building would make an interesting study, and it is to be hoped that some one to whom they are accessible will analyze them. Unless its partitions and walls were of a very unusual type I do not think they gave much help to the columns and beams at the apex of the storm. If its interior beams did not run parallel to the wind, or it did not have solid concrete spandrel walls, a miracle has taken place.

In conclusion, no formula, and no rule of thumb can take the place of careful analysis of wind stresses in all except a very few buildings.

DISCUSSION.

Mr. Foster. C. B. FOSTER.—What consideration do you give to dead weights? In my estimation a dead weight of a 20 or 15-story building, even, is almost sufficient to overcome any wind pressure, the inertia of the dead weight.

Mr. Smith. ALBERT SMITH.—If I understood your question, you feel that if I had pin joints here, there and there in all the columns and all the beams, the fact that the building carried a very large load would cause it to stand up?

Mr. Foster. MR. FOSTER.—My idea is that if there are 6,000,000 lb. of dead weight and you claim that the total wind pressure is 1,000,000 lb., can 1,000,000 lb. move 6,000,000?

Mr. Smith. MR. SMITH.—You mean that if I had pin joints at every beam and column and put enough weight on it, it would stand. Of course it would take an enormous force to slide 6,000,000 lb. initially.

Report of Tests on 300-ft. Reinforced-Concrete Chimney.

By Earle D. McKay.*

A complete description of the 300-ft. reinforced-concrete chimney built by the Universal Portland Cement Co. at its Duluth plant, together with the proposed series of tests and the necessary apparatus therefor, was given by Benjamin Wilk at the 1926 convention of the American Concrete Institute.

A sub-committee on Tests and Research of Committee S-1 on Chimneys, acting in an advisory capacity, outlined these tests to determine:

I. Temperature gradient through the lining and wall of a reinforced-concrete chimney.
II. Nature and intensity of stresses developed in concrete and steel reinforcing due to wind pressure upon the chimney and as a result of flow of heat through the wall.
III. Wind velocity adjacent to the chimney.
IV. Distribution of the pressure developed on the cylindrical surface of a chimney by wind.
V. The nature and extent of the deflection of the chimney due to wind pressure and heat.

The construction of the chimney on which these tests are being conducted was started in April, 1925, and completed the first week in January, 1926. This chimney was designed to serve five rotary clinker cement kilns, inlets for the gas being placed at two elevations in the chimney shell at right angles to each other.

On Jan. 22, 1926, this chimney was first placed in operation with gas from one kiln entering the upper breeching. The lower breeching was not yet closed, thus permitting the outside air to enter the chimney at this point. At various times later other kilns were added to the service and on March 19 the lower breeching was closed and the entire volume of gas from the plant was passed through the chimney.

It is estimated that at full operation approximately 500,000 cu. ft. of gas per minute is introduced into the chimney. The temperature in the burning zone of the kilns is approximately 2,800 deg. F. and the gas first passes through a dust-collecting and cooling system, entering the chimney at approximately 500 deg. F.

* Universal Portland Cement Co.

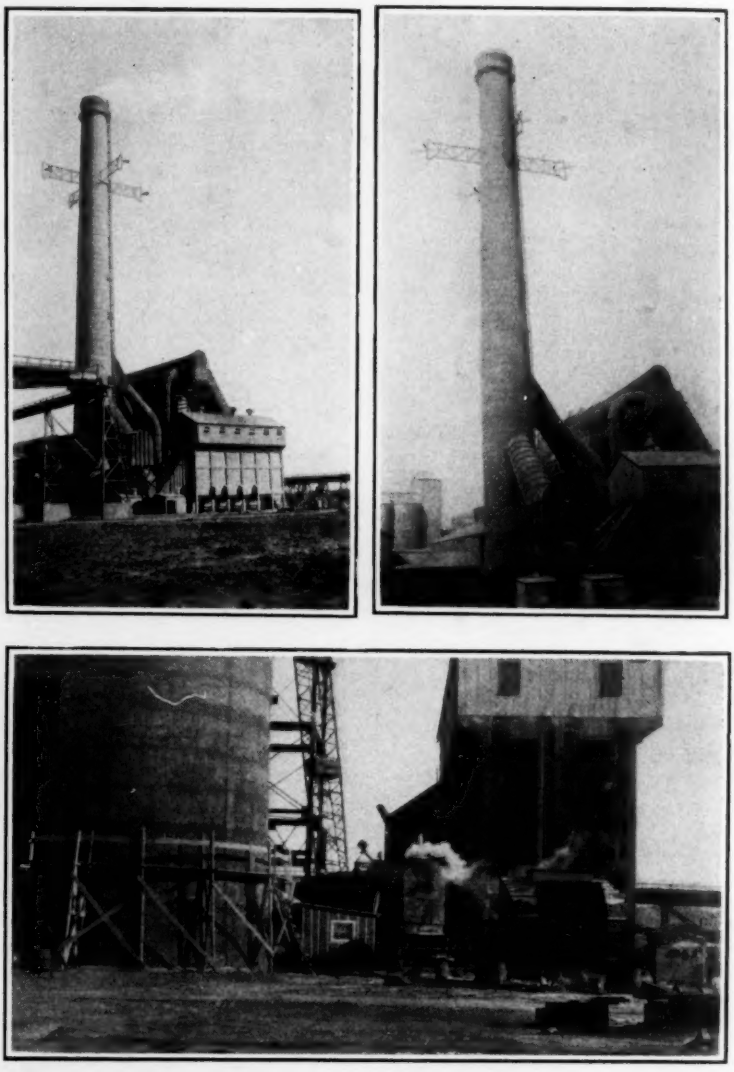


FIG. 1.—THREE VIEWS OF THE COMPLETED CHIMNEY.

The sample of the gas taken on Jan. 22, 1926, was analyzed as follows:

	Per Cent
CO_2 (Carbon Dioxide)	22.2
O (Oxygen)	3.5
CO (Carbon Monoxide)	0
N (Nitrogen)	74.3
	100.0

Fig. 1 shows the chimney in full operation.

These pictures give a general idea of the location and arrangement of the testing equipment. The trusses supporting the outriggers and platforms used for taking strain-gage readings can be readily seen. The line of pressure tubes around the circumference of the chimney can be detected midway between the upper platform and the outriggers. The shack at the base of the chimney in which the instruments are housed can be seen on the right.

Due to a combination of circumstances, the completion of the test apparatus was greatly delayed and it was not until Oct. 7, 1926, that the first complete series of tests was made. Since that time, however, regular tests have been run on a schedule calling for complete readings every two or three weeks.

This report is not intended to be final, for it is the plan to continue these tests throughout the year 1927, and longer if necessary. The results given must, therefore, be treated as of a preliminary nature.

I.—Temperature.

In obtaining the temperature gradient through the chimney shell 30 thermocouples, located at five different elevations, were used and were so wired as to permit the taking of readings with a potentiometer housed at the base of the chimney.

These elevations, with reference to the structural details of the chimney, are as follows:

A. Just below the lower breeching;
B. Just below the upper breeching;
C. Just below the top of the lining;
D. Just above the top of the lining;
E. Near the top of the chimney.

The locations of these thermocouples with reference to the chimney shell as well as to the ground elevation are shown in detail in Fig. 2.

The method of placing the thermocouples was to insert the "hot junction" through a brass sleeve in the concrete until it came in contact with the latter. The next step was to pack the tube with asbestos and seal it.

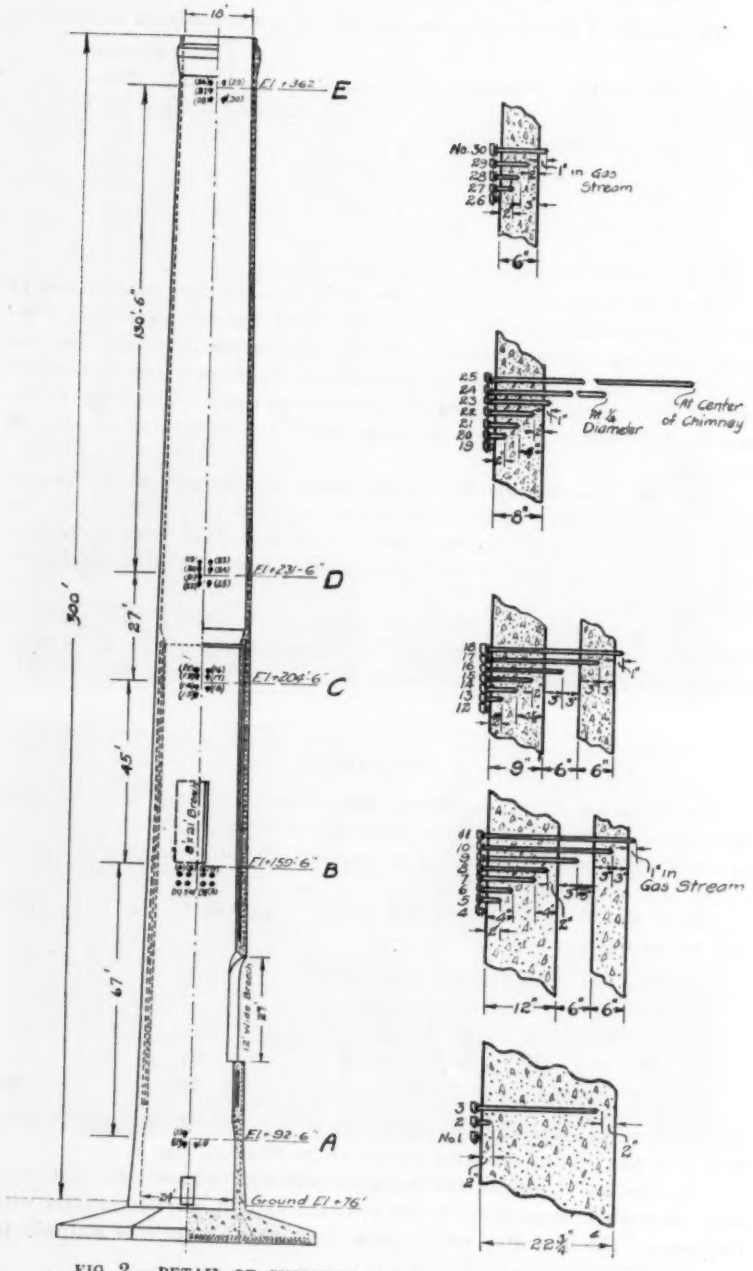


FIG. 2.—DETAIL OF CHIMNEY AND THERMOCOUPLE LOCATIONS.

Before readings were started the potentiometer was calibrated to read directly in degrees fahrenheit.

Temperature readings were begun on Jan. 21, 1926, just 28 hours before the gas was turned into the chimney, and were continued hourly for 100 hours. The time required for taking a complete set of readings was approximately 5 min. During this first test period the lower breeching was still open admitting cold air into the chimney. Occasional readings were taken from that time until the middle of March, when work was begun closing the lower breeching. At this point more frequent readings were taken. Several sets of readings were taken during the early summer months. Observations were resumed in the Fall when temperature conditions were fairly well stabilized. A special series of readings was taken on Dec. 24 to 27, 1926, to observe the change in conditions caused by a shut down of the mill. The results here obtained show the drop in temperature after heat is turned off and a building up again upon resumption of operations.

All of the above temperature readings are plotted in Fig. 3, which is divided into five sections corresponding to the five thermocouple locations.

You will note from a study of Fig. 3 (d) for El. "D" that the temperature of the concrete and the temperature of the air follow each other quite closely during the period preceding admission of the gas into the chimney. The temperature of the concrete follows in general the temperature of the outside air, but the temperature on the inside of the chimney does not increase as rapidly.

Upon admission of the gas the temperatures in the stream rise suddenly 35 deg. It will be noticed that throughout the entire early test period the temperature of the concrete is represented by a line much smoother than that of the gas within or of the air without. Sudden rises in temperature of the gas for short periods of an hour or so produced no apparent increase in the concrete temperature, but for long periods the high temperatures produced some effect.

During the month of March there is a rise in the temperature of the gas within the chimney. This is due partly to the closing of the south upper breeching and the lower breeching. There is a continued general rise, however, beyond this point which is apparently due to change in the operating conditions, since the outside air shows a general level throughout this period. Here the effect of the increase in temperature of the gas is to produce a warming up of the chimney. A final increase of 60 deg. from March 19 to April 4 produced an increase of about 40 deg. in the center of the concrete, outside air temperatures on both dates being the same.

The curves obtained during the months of October, November and December show among other things, the remarkable temperature conditions produced by a shut-down of the mill. On Oct. 7 the temperature at the center of the gas stream was 366 deg., the corresponding temperature of the concrete on the inside of the shell was 150 deg., the temperature of the outside air was 50 deg. One day later the temperature of the gas

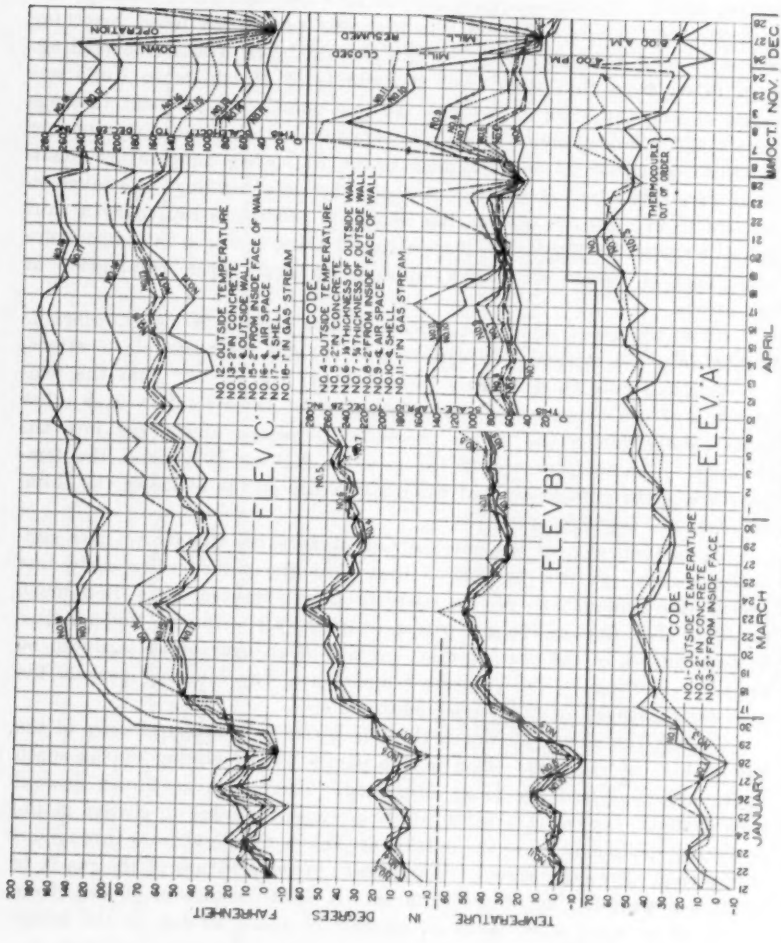


FIG. 3 (PART 1).—TEMPERATURE READINGS.

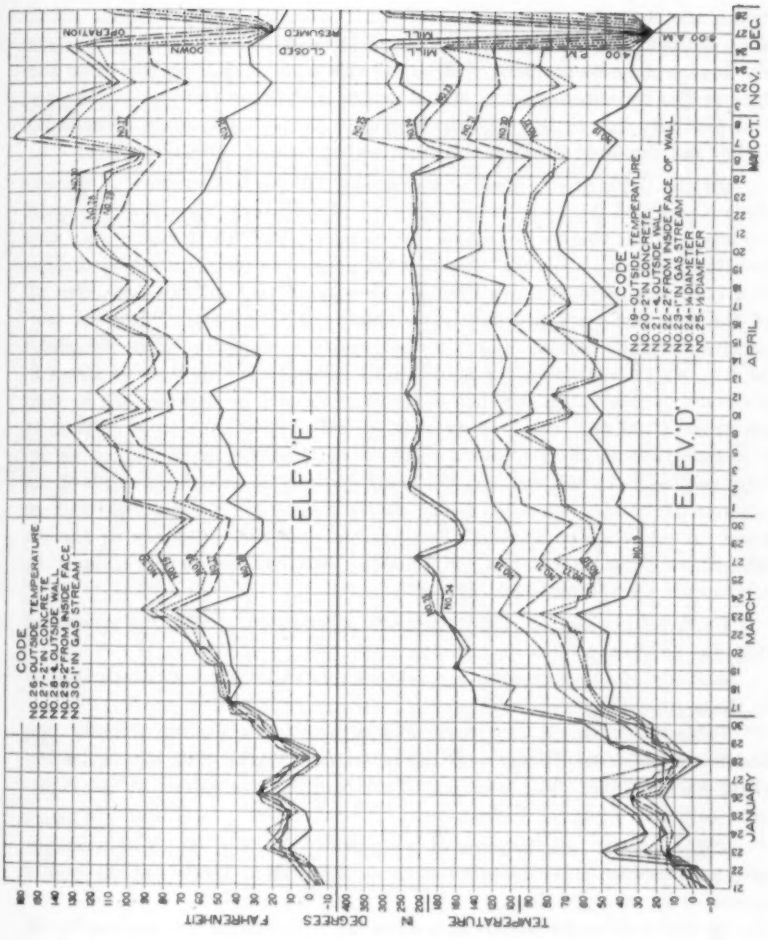


FIG. 3 (PART 2).—TEMPERATURE READINGS.

was 349 deg., the concrete 140 deg., and the air 56 deg. The temperature of the gas one inch from the face of the concrete was 54 deg. above the temperature of the concrete at the center of the shell, and 155 deg. less than the temperature in the center of the stream. It may thus be seen that at this particular point a difference in temperature of 210 deg. between the center of the gas stream and the center of the concrete exists. This difference is here more pronounced than at the higher elevations and the reason may be partially due to the effect of the jog on the inner surface at the top of the lining, which causes a deflection of the gas stream, and partially due to the cold air drawn up through the space between the lining and the shell through the vents at the base of the lining.

Before the mill was shut down on Dec. 24 the temperature of the gas stream was 288 deg., and after this, the temperature began to drop quite rapidly. It continued to fall off for 2 hours, and at 10 hours was still showing a gradual decline. Even at 24 hours the temperature of the gas was still considerably above that of the outside air, but one day later all of the readings were practically the same. The maximum drop of temperature of the concrete was 150 deg., and the minimum drop was about 50 deg., which would cause a condition of strain in the shell. The stress in the concrete due to an internal adjustment caused by a difference in temperature of 100 deg., using an average coefficient of expansion of .000006 at a temperature of 70 deg. and a modulus of elasticity of 5,800,000, as determined from the test cylinders, is much beyond the tensile strength of the concrete if no deflection or distortion is assumed. No attempt will be made at this time to analyze the probable condition of stress as the coefficient of expansion of test cylinders at different temperatures must be determined for this purpose.

Upon resumption of mill operations a condition almost the reverse of that due to closing down is noted.

A study of the curves for the other levels will show conditions similar to those just discussed.

Table I shows all temperature readings taken during October and November when conditions were fairly stable and the mill was operating under normal load. The averages of all readings are shown in the right-hand column.

These results are illustrated graphically in Fig. 4. The averages have been plotted on chimney sections at the various levels and the points connected by straight lines.

In Fig. 5 these same averages have been plotted at the various levels for the height of the chimney. The points occupying the same position relative to the chimney shell have been connected by straight lines.

Table II is the summary of temperature differences in the concrete of the chimney shell made up from the same readings contained in Table I.

It will be seen from these curves that the temperature through the lining and wall varies approximately as a straight line. The average drop

in temperature measured for the six-inch thickness at the "E" level is 20.5 deg. per inch of width, while the maximum is 26.5 deg. It will be noted that this point is above the lining. At "C" level, which is below the top of the lining, the average drop in temperature was 8.1 deg. per inch of width and the maximum 10.6 deg.

TABLE I—TABULATION OF TEMPERATURE READINGS, OCT. 7 TO NOV. 24, 1926

Thermo-couple No.	Oct. 7, 1926	Oct. 8 1926		Nov. 3, 1926					Nov. 23, 1926					Nov. 24, 1926	Average all Readings
	3:35 P.M.	11:30 A.M.	3:00 P.M.	9:45 A.M.	12:00 Noon	12:40 P.M.	1:15 P.M.	4:15 P.M.	9:30 A.M.	10:00 A.M.	12:00 Noon	1:30 P.M.	4:15 P.M.	9:00 A.M.	
1.....	50	60	63	35	36	36	37	40	28	28	30	31	33	24	38°
2.....	70	75	76	42	40	43	45	45	37	40	38	40	40	33	47°
3.....	87	85	88	72	71	75	78	81	78	76	77	77	78	73	78°
4.....	50	54	55	35	37	37	32	39	25	27	25	30	28	27	36°
5.....	85	85	84	51	51	55	57	58	46	48	50	45	50	59	59°
6.....	92	91	99	43	53	55	58	46	50	51	46	48	49	60	60°
7.....	117	115	117	68	67	72	75	77	70	73	66	62	68	72	80°
8.....	128	126	130	132	122	115	97	88	55	57	50	53	53	67	91°
9.....	147	144	150	145	137	120	112	100	100	102	95	95	96	98	117°
10.....	151	248	237	188	195	187	188	182	183	185	175	177	175	176	189°
11.....	281	273	273	218	210	197	193	220	202	202	194	198	194	190	218°
12.....	50	56	56	35	38	38	37	41	26	28	25	30	28	37	38°
13.....	85	84	84	60	63	65	66	67	51	52	49	53	48	60	63°
14.....	92	91	92	66	70	71	72	73	61	60	56	61	57	73	71°
15.....	138	128	128	98	102	100	103	103	95	95	88	93	89	93	104°
16.....	162	153	154	118	123	123	127	123	116	118	108	115	110	113	126°
17.....	252	242	242	217	218	217	215	212	207	208	197	202	197	197	216°
18.....	276	267	267	252	253	233	222	250	233	233	228	258	225	220	248°
19.....	49	56	57	35	38	39	38	40	27	28	25	31	30	38	38°
20.....	106	104	104	92	102	95	98	95	77	77	68	75	67	86	89°
21.....	150	140	41	135	137	138	137	133	122	122	110	113	107	118	129°
22.....	91	97	99	83	88	87	87	85	67	68	63	63	63	86	81°
23.....	208	194	195	197	188	176	172	195	170	170	160	161	156	157	179°
24.....	215	217	220	168	170	113	217	250	250	250	270	270	265	250	223°
25.....	366	349	343	378	317	247	233	333	295	293	305	305	298	268	309°
26.....	43	48	50	27	30	30	32	33	23	25	22	26	26	35	32°
27.....	102	102	105	88	92	92	93	90	74	73	68	68	68	88	86°
28.....	134	129	132	127	119	120	120	117	102	101	87	90	92	113	113°
29.....	155	142	145	133	132	133	130	130	120	120	105	107	108	115	127°
30.....	167	155	159	192	142	136	135	145	124	123	110	112	113	123	138°
Outside Temp.	62°	67°	67°	15°	15°	15°	15°	15°	23°	23°	23°	23°	23°	22°	29°
Room Temp.	68°	70°	70°	65°	65°	65°	65°	65°	30°	30°	30°	30°	30°	32°	51°
Weather	25 mi. N.E. Wind Cl'dy	30 mi. N.E. Wind Cloudy		10 mi. N.E. Wind Cloudy					10 mi. N.E. Wind Clear					10 mi. N. Wind Clear	
				* Mill closed down 12 Noon to 1:00 P.M.											

These values are slightly higher than those reported by E. A. Dockstader at the 1925 convention of the American Concrete Institute. The results, however, are not directly comparable because of the differences in size of the chimney, temperature of the outside air, and temperature and rate of flow of gas through the stack.

II.—Strain Gage Readings.

In a study of the stresses from strain gage readings on an exposed concrete structure, a number of factors affecting the deformations must be given consideration, such as temperature, radiant heat, moisture, live and dead load, plastic flow and shrinkage. Measurements taken on the chimney to date have been made to determine the character and amount of stress in the steel due to these factors. These results will be of particular use in the study of the stresses due to wind load, which are to be obtained later with recording instruments. To date no records of deformations due to wind have been obtained, but it is the plan to install this recording equipment this winter and complete this study before the next meeting. The measured deformations taken with the Berry strain gage are shown in Fig. 6.

TABLE II.—TEMPERATURE DIFFERENCES IN CHIMNEY SHELL

Thermo-couple	Distance between Couples, in.	Grand Average		Maximum		Minimum	
		Total Temperature Difference	Difference per in.	Total Temperature Difference	Difference per in.	Total Temperature Difference	Difference per in.
5–6	2	4.8	2.4	15	7.5	0	0
6–7	4	20.0	5.0	31	7.7	12	3.0
7–8	2	21.9	11.0	64	32.0	5	2.5
13–14	2½	7.7	3.1	13	5.2	6	2.4
14–15	2½	32.7	13.1	46	18.4	20	8.0
20–21	2	39.8	19.9	45	22.5	32	16.0
21–22	2	48.0	24.0	59	29.5	32	16.0
27–28	1	27.1	27.1	39	39.0	19	19.0
28–29	1	13.7	13.7	21	21.0	2	2.0
2–3	18¾	30.9	1.6	41	2.2	10	0.5
5–8	8	32.1	4.0	81	10.1	0	0
13–15	5	40.5	8.1	53	10.6	33	6.6
20–22	4	8.5	2.1	15	3.7	0	0
27–29	2	41.0	20.5	53	26.5	27	13.5

These results represent the actual deformations in the steel and concrete at the elevations indicated on the drawing. No corrections have been made for temperature nor moisture, so that the curves represent the actual movements which have taken place in the gage lines since the readings were started.

A comparison of the movements of the vertical and the horizontal gage lines is of particular interest because of the fact that temperature effects, moisture effects, etc., are alike for both readings. Any difference in the deformations measured must, therefore, be accounted for by load conditions. Shrinkage should be about the same in both directions. In the curves representing the actual movements that have taken place in the gas lines since the initial set of readings, two groups showed both longitudinal and vertical deformations. The difference between the horizontal and vertical movements represents the actual net deformation due primarily to plastic flow of the concrete under load. The dead load on the chimney is the same at this time as at the time readings were started,

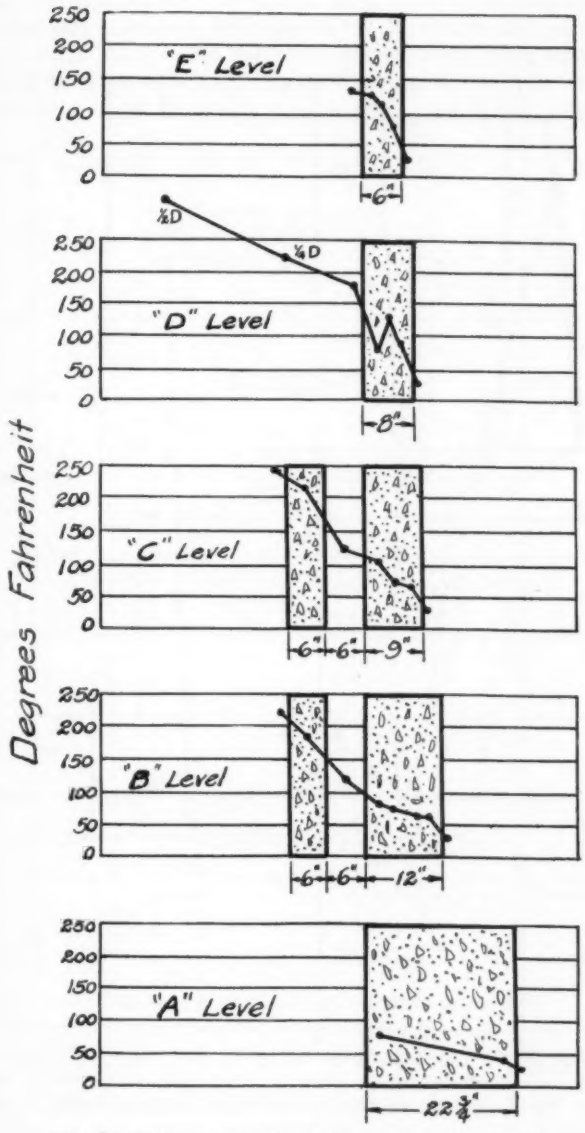


FIG. 4.—TEMPERATURES THROUGH CHIMNEY SHELL.

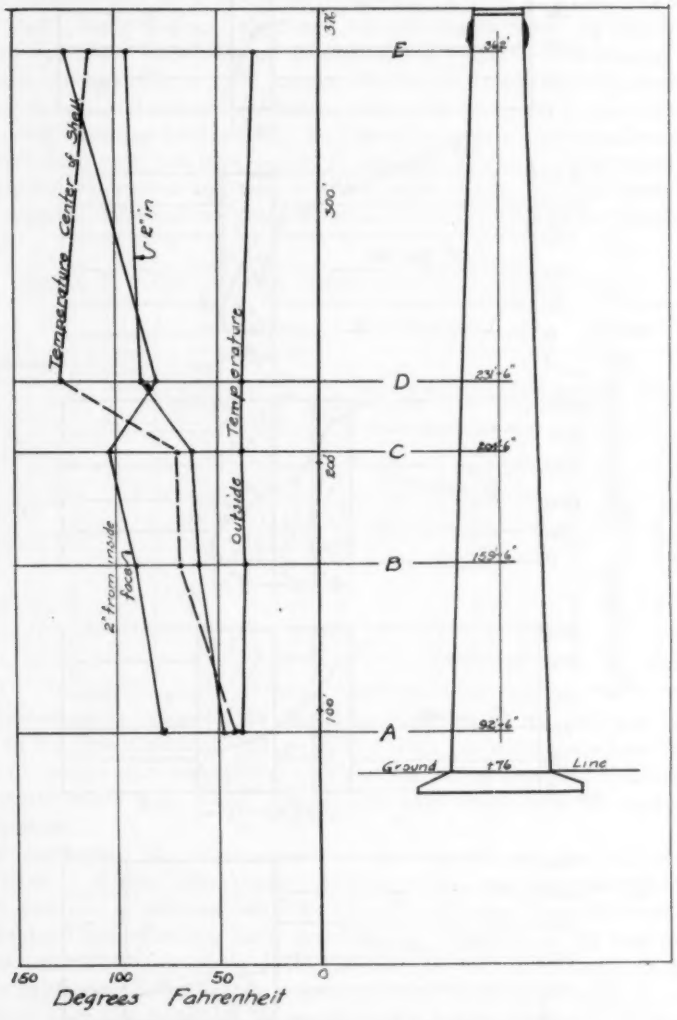


FIG. 5.—TEMPERATURE READINGS ON VERTICAL LINE.

with the exception of the effect produced by the weight of the intake flues. This load is comparatively very small. Some slight effect may also be due to the fact that there is a difference in the horizontal and vertical reinforcement which would resist the action of shrinkage. This movement, as can be seen from the curves at El. 79 and El. 92, is relatively quite small and indicates that the plastic flow under dead load is very slight. This

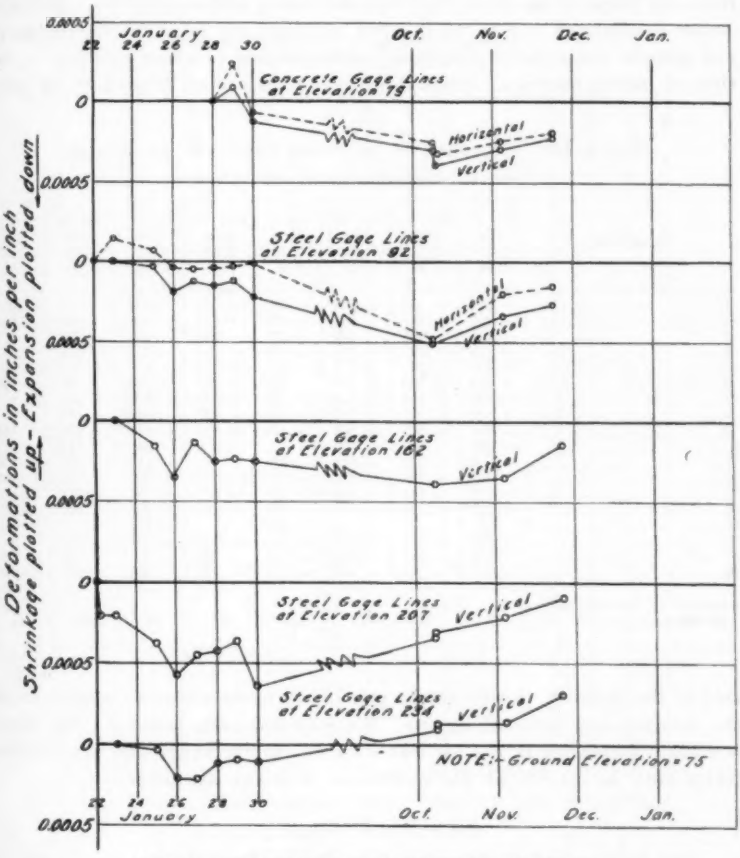


FIG. 6.—DEFORMATIONS IN CONCRETE CHIMNEY.

may be partially accounted for by the high elastic properties of the concrete as indicated by the cylinder tests, since values of the modulus of elasticity varied from 5,600,000 to 5,950,000 at the age of 1¼ years, and the curves are essentially straight lines up to one-half of the ultimate strength. Beyond this point there is only a slight curvature. From a study of Fig. 6 it will be seen that there is a general similarity of the

curves throughout. The rise and fall is probably due to seasonal changes in the atmosphere. There has been an expansion in most of the gage lines from the initial reading in January, 1926, and the present time, but the expansion is becoming less and the movement is now very nearly what it was a year ago. There has been a general shortening of all of the gage lines during the months of October and November, and it seems probable from the shape of the curve that this shortening will continue for some time. Better comparison can be made when readings are taken at temperatures and general atmospheric conditions corresponding to those existing at the time of initial readings. Since these gage lines are all located on the out-

TABLE III.—TABULATION OF WIND PRESSURE READINGS

(Corrected and reduced to vertical inches of water)

No. of Tube	Dec. 15, 1926 2:05 P.M.	Dec. 15, 1926 2:30 P.M.	Dec. 15, 1926 1:55 P.M.	Oct. 7, 1926 3:55 P.M.	Dec. 14, 1926 3:45 P.M.	Oct. 8, 1926 4:00 P.M.	Dec. 14, 1926 4:00 P.M.	Oct. 8, 1926 10:00 A.M.
1	−.225	−.173	−.094	−.163	−.244	−.134	−.134	−.005
2	−.379	−.398	−.268	−.178	−.479	−.138	−.258	−.034
3	−.515	−.445	−.293	−.135	−.394	−.094	−.203	−.021
4	−.559	−.237	−.118	−.157	−.138	−.118	−.058	−.013
5	−.046	+.220	+.230	−.116	+.158	−.073	+.210	+.023
6	+.369	+.449	+.389	−.159	+.165	−.099	+.260	−.040
7	+.621	+.445	+.405	−.147	+.070	−.098	+.345	0
8	+.480	+.091	+.081	−.137	−.177	−.087	+.051	0
9	−.020	−.358	−.258	−.174	−.480	−.178	−.208	−.024
10	−.568	−.568	−.376	−.306	−.548	−.266	−.246	0
11	−.746	−.324	−.186	−.275	−.315	−.146	−.087	+.010
12	−.388	−.238	−.168	−.088	−.334	+.050	−.098	+.005
13	−.346	−.228	−.168	+.109	−.307	+.191	−.098	+.150
14	−.346	−.216	−.167	+.286	−.296	+.246	−.097	+.035
15	....	....	....	+.151	....	+.172	....	+.015
16	−.305	−.196	−.137	+.110	−.315	−.078	−.098	0
17	−.336	−.246	−.166	−.166	−.286	−.206	−.086	−.022
18	−.315	−.226	−.145	−.077	−.276	−.067	−.067	−.010
Anemometer Reading, miles per hour	42	35	34	30	27	25	19	10

side of the chimney, it will not be possible to make a direct comparison of the internal and external strains. We may conclude, however, that there is some compression in the vertical steel due to shrinkage and plastic flow which must be considered when problems of design are taken up.

III.—Wind Velocity.

Two independent studies of wind velocities were made. In the first a series of pressure pipes leading to manometer tubes were used, and in the second a Burton recording anemometer was employed. In taking simultaneous manometer readings it was at first intended to photograph the panel containing all of the tubes. A more satisfactory method, however, was devised which consisted of pinching the rubber connections directly above the manometer tubes by means of a steel clamp. The Burton anemometer was self-recording and the tie between this instrument and the

manometer readings was effected by making a pencil mark on the chart at the time the clamp was closed. Ranges in velocity from 0 to 60 miles per hour were observed at various times.

The pressures in the outrigger impact tubes were considerably less than was expected. The reason for this is not yet known, and further tests of the apparatus will be necessary before this condition can be explained.

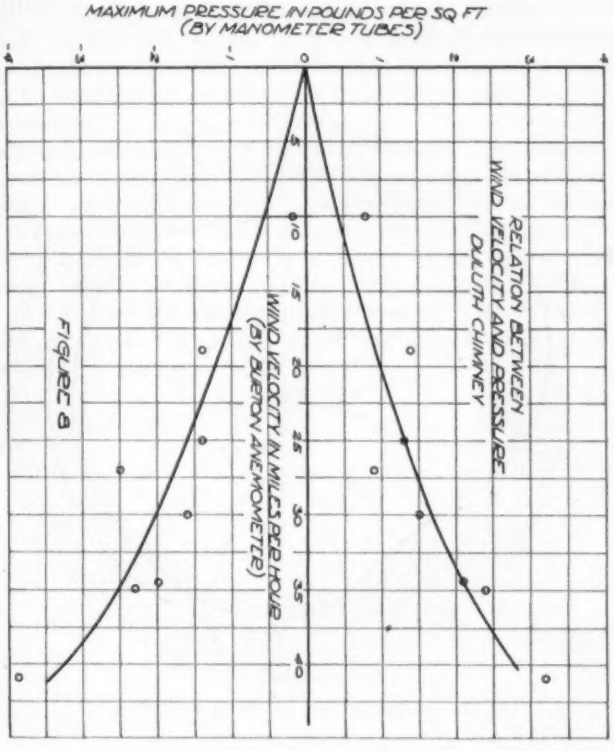


FIG. 8.—RELATION BETWEEN WIND VELOCITY AND PRESSURE.

IV.—Wind Pressures.

Wind pressures were read by means of impact pressure tubes at 18 points equidistant around the chimney at El. 265. The openings into the tube are flush with the face of the chimney and transmit the pressure in separate ⅜-in. pipes to the base where the pressures are read. Glass tubes filled with colored liquid are placed side by side on a tilting table. The table is so arranged that the angle the tubes make with the horizontal can be varied so as to give any multiplication of the readings desired. Thus

far all readings have been taken with the manometer table inclined at a slope of 1 to 10, which multiplies the readings by 10. The results of a group of the readings are shown in Table III.

The maximum pressures obtained from these readings are plotted in Fig. 8.

The curve represents the average maximum pressure for velocities up to 42 miles per hour. Since these readings are taken with two different types of instruments, in which experimental errors may be additive, the points are somewhat at variance. The curves indicate, however, an increase

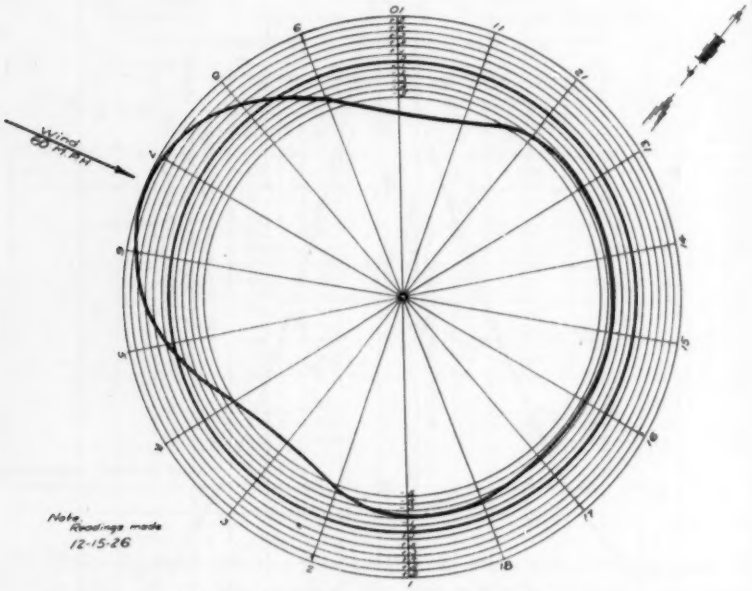


FIG. 9.—DISTRIBUTION OF WIND STRESSES.

in the positive and negative pressures as the wind velocity increases up to 40 miles per hour. Sufficient data has not been obtained beyond this velocity to warrant extending the curves.

Fig. 9 represents the distribution of pressures around the stack. It can be seen that the pressure is positive on three-fifths of the projected circumference on the windward side of the stack, and negative on the balance of this side. It is negative throughout on the leeward side. The maximum positive pressure occurs at the point directly into the wind and the maximum negative pressure occurs also on the windward side of approximately 70 deg. around the stack on either side of the maximum positive pressure.

V.—Deflections.

An attempt was made to measure the deflection of the chimney with the apparatus proposed by the committee. This consisted of a telescope and mirror mounted on an isolated foundation on the ground 44 ft. from the base of the chimney and a horizontal checker board plate fastened firmly to the chimney near the top. It was found, however, that the vibration of the concrete base supporting the telescope, due to the operation of machinery in this vicinity, was so great as to make accurate observations impossible. In addition, the type of mirror used appeared to reduce the clearness of vision considerably. This method of measuring deflection will probably be unsuited for readings in a severe storm, since a clear atmosphere is essential.

Acknowledgments are due Ray S. Huey, general superintendent of the Duluth mill for co-operation in obtaining these results, and to Fred Robinson, assistant general superintendent, under whose direction the test apparatus was installed. Particular credit must given M. B. Lagaard, formerly assistant professor at the University of Minnesota and now with the Portland Cement Association, Chicago, whose help has been invaluable in obtaining and interpreting the strain gage readings. H. E. Bernt, G. W. Mork, and R. Conrad Cooper have aided very materially in the collection and tabulation of this data.

THE REINFORCED-CONCRETE COLUMN.

By Phil. J. Markmann*

Stresses

The well-known rational equation expressing the apportionment of the unit load $\frac{P}{A}$ between steel and concrete in conformance with the relative compressibilities of the two materials is:

$$\frac{P}{A} = f_c[1 + (n-1)p] \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

reduced from its expanded form:

$$\frac{P}{A} = f_c(1-p) + nf_cp \quad \ldots\ldots\ldots\ldots\ldots\ldots (1^a)$$

where f_c = the unit compressive stress of the concrete, and $nf_c = f_s$ = the unit compressive stress of the steel.
From eq. 1 we have

$$f_c = \frac{\frac{P}{A}}{1 + (n-1)p} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

and

$$f_s = \frac{n}{1 + (n-1)p} \times \frac{P}{A} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

If E the "modulus of elasticity" of the steel, and, therefore

$\frac{E}{n}$ the "modulus of elasticity" of the concrete, and if m is the unrestrained unit contraction of the completely hardened concrete, then

$$f_h = -\frac{E}{n} \times m \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

(the — sign used to denote "tension")

is the unit stress (tension) of the concrete when the latter is fully restrained from contracting during hardening. The column of plain concrete offers no resistance to the contraction and, therefore, there is no tension in the concrete. In the reinforced concrete column the steel, being bonded to the concrete,

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by its compressive strength *tends* to resist the contraction of the concrete, but as the steel cannot be put in compression without engaging the concrete to cooperate with it, in conformance with eq. 1, the resistance to the contraction of the concrete is the resistance offered *by the steel in combination with the concrete.* This combined resistance is *the resistance by the reinforced concrete column. The reinforced concrete column, not the steel alone, resists the contraction of the concrete.* The unit size (1 sq. in.) reinforced concrete column bears a load of

$$\frac{P_i}{A} = \frac{E}{n} \times m\,(1-p) \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

This is *the unit internal load.*

The unit compressive concrete stress f_{ci} due to this internal load follows from eq. 2 by substituting $\frac{E}{n} \times m\,(1-p)$ for $\frac{P}{A}$ we have

$$f_{ci} = \frac{1-p}{1+(n-1)p} \times \frac{E}{n} \times m \ldots\ldots\ldots\ldots (6)$$

The *unit compressive steel stress* due to this internal load follows from eq. 3 by the same substitution, reducing, we have

$$f_{si} = \frac{1-p}{1+(n-1)p} \times E \times m \ldots\ldots\ldots\ldots (7)$$

The *actual unit concrete stress* is the *resultant* of f_h (eq. 4), the unit tensile concrete stress due to *fully* checked contraction (the anti-contractive tension), and the unit compressive stress f_{ci} (eq. 6) received from the unit internal load $\frac{P_i}{A}$ (eq. 5), we have

$$f_{cr} = f_h + f_{ci}$$

and substituting for f_h and f_{ci} their values from eq. 4 and 6, and reducing, we have

$$f_{cr} = -\frac{p}{1+(n-1)p} \times E \times m \ldots\ldots\ldots\ldots (8)$$

This resultant unit concrete stress f_{cr} is always a tensile stress [except when $p=0$ (plain concrete) when it becomes zero], it is the *residual unit tension* of the concrete, to which it has been reduced from its potential value $f_h = -\frac{E}{n} \times m$ (eq. 4) by the back-action of the internal load. At any stage of the hardening process, as well as at completed hardening, the total concrete stress and the total steel stress in the column are in equilibrium, the *tension* in the concrete of the unit size (1 sq. in.) reinforced concrete column balances the *compression* in the steel, so that

$$f_{cr}\,(1-p) + f_{si} \times p = 0$$

Substitution of the values of f_{cr} and f_{si} (from eq. 8 and 7) satisfies this equation and proves the correctness of the values of f_{cr} and f_{si}.*

* Another derivation of f_{cr} and f_{st} is given in Appendix I.

When hardening is completed, the anti-contractive unit tension of the concrete (eq. 4) together with the action of the internal load (eq. 5), considered by themselves (without any external load), have brought the concrete to a state of *tensile stress* f_{cr} (the residual tension given by eq. 8), and the steel to a *compressive stress* f_{si} (given by eq. 7). This *relative* condition of stress in the two materials, is bound to be changed with the coming of external load. As the water seeks its level, so the concrete seeks to put itself into elastic equilibrium with the steel. This equilibrium *demands* that the concrete stress become a compressive stress equal to $\frac{1}{n}$ of the compressive stress f_{si} in the steel, the latter due to the internal load. This equalization *can be* and *is* effected by the concrete *alone*, for a time, taking the pressure of the external load $\frac{P_e}{A}$. The whole external load P_e (the weight of the column-stack, floors and roof above the column in question, and may be other superimposed weights), in concrete construction, is always primarily and directly placed upon the concrete area $A(1-p)$ of the column and a part of it is subsequently transferred to the steel *when* and *as* the elastic equilibrium demands it. The steel will not demand any of the external load before the concrete stress will have been increased from its residual tension f_{cr} (eq. 8) to zero and from there up to $\frac{1}{n}f_{si} = f_{ci}$ (given by eq. 6). During this period all of the oncoming external load stays on the concrete (on which it is placed), the steel stress remains *stationary* at f_{si} (the stress put into it by the internal load). Or, in other words, the steel is *neutral* during this period. When and after the concrete has saturated itself (from the external load) up to the amount given by eq. 6, all further external load is *apportioned* between concrete and steel in conformance with eq. 1.—The external load will, therefore, divide itself into two (2) parts, viz: a *first* part all retained by the concrete and, therefore, resting upon the concrete area $A(1-p)$ the same area which is one of the factors of the internal load (see eq. 5), and a *second* part which *apportions* itself between the concrete and the steel. The *first* part of the whole external load P_e, temporarily designated as P'_e, and introducing a coefficient k, is

$$P'_e = k \times P_e$$

therefore, $\frac{P'_e}{A(1-p)} = \frac{k \times P_e}{A(1-p)}$ is the pressure on the sq. in. of concrete area, or expressing this pressure in terms of $\frac{P_e}{A}$ we have the unit pressure upon the concrete by this first part of the external load:

$$f_{c1} = \frac{k}{1-p} \times \frac{P_e}{A} = -f_{cr} + f_{ci}$$

the first term of the right member is the unit pressure which brings the concrete from its residual tension up to zero stress, the second term is the

additional unit pressure which brings the concrete from zero stress up to $\frac{1}{n} f_{si} = f_{ci}$. Substituting for f_{cr} and f_{ci} their values from eq. 8 and 6, and reducing, we have

$$f_{c1} = \frac{k}{1-p} \times \frac{P_e}{A} = \frac{E}{n} \times m \ldots\ldots\ldots\ldots (9)$$

Eq. 9 shows the unit pressure upon the concrete by the *first* part of the external load $\frac{P_e}{A}$ to be *numerically* equal to the anti-contractive tensile stress f_h (eq. 4). Their sum = 0.

Digressing from our objective—the division of the external load into its two parts—for the purpose of defining the unit concrete *stress*: after the unit *pressure* $f_{c1} = \frac{E}{n} m$ by the *first* part of the external load has been applied to the concrete, the unit stress becomes the *resultant* of this f_{c1} and the residual tensile stress f_{cr} existing prior to the application of the external load.

As

$$f_{c1} = -f_h \text{ (see eq. 4 and 9)}$$

and

$$f_{cr} = f_h + f_{ei} \text{ we have}$$

$$\therefore f_c = f_{c1} + f_{cr} = f_{ci} \text{ as the unit concrete}$$

stress at the moment of attained elastic equilibrium.

Continuing the interrupted development: The unit pressure by the first part of the external load, eq. 9, annuls the unit anti-contractive tension f_h (eq. 4). By eq. 9

$$k \times \frac{P_e}{A} = \frac{E}{n} \times m(1-p) \ldots\ldots\ldots\ldots (9^a)$$

This amount of external load (eq. 9^a) terminates the elastic disparity between concrete and steel, it may be called the "compensating" load. This compensating (external) load

(1) adds the unit pressure $\frac{E}{n} m$ to the residual tension f_{cr} of the concrete, giving the *resultant* compressive stress f_{ci}

(2) adds nothing to the steel stress f_{si}

(3) adds nothing to the total shear $-f_{cr}(1-p = f_{si} \times p$ between the concrete and the steel in the half length of the unit size $(1 \square'')$ reinforced column, because none of this load is transmitted to the steel.

The combined effects are:

concrete stress $-f_h + f_{cr} = f_{ci}$ (see above)

steel stress $f_{si} + 0 = f_{si}$

the total shear $\left.\begin{matrix} -f_{cr}(1-p) + 0 \\ \text{or } f_{si} \times p + 0 \end{matrix}\right\} = -f_{cr}(1-p = f_{si} \times p$

Both, the unit steel stress f_{si} and the total shear in the half length of the

unit size (1 □″) reinforced column, remain *constant* during the gradual application of the *first* part $\left[=\frac{E}{n}m\ (1-p)\right]$ of the external load, in other words, up to the moment of attained elastic equilibrium, the associated stresses then being f_{ci} and $n \times f_{ci} = f_{si}$ in the concrete and steel, respectively.

The remainder, or the second part, of the whole external load is, therefore,

$$\frac{P_e}{A} - \frac{kP_e}{A} = \frac{P_e}{A} - \frac{E}{n} \times m(1-p) \ldots\ldots (10)$$

This *remainder*, as has been said above, is *apportioned* between the concrete and the steel conforming to eq. 1, because the elastic equilibrium is *now* established. Substituting this second part of the external load (eq. 10) for $\frac{P}{A}$ in eq. 2, we have

$$f_{c2} = \frac{\frac{P_e}{A}}{1+(n-1)p} - \frac{\frac{E}{n} \times m(1-p)}{1+(n-1)p} \ldots\ldots\ldots\ldots (11)$$

This is the unit compressive concrete stress due to the *second* part of the external load.

Joining the three concrete stresses f_{cr} (the resultant of f_h and f_{ci}), f_{c1} and f_{c2} into a force polygon (see Diagram 4) we find f_c (closing the polygon) is their *resultant*.

Or, algebraically, we have

$$f_c = f_{cr} + f_{c1} + f_{c2} \ldots\ldots\ldots\ldots\ldots (12)$$

substituting the values of the three (3) terms of the right member, and reducing, we have

$$f_c = \frac{\frac{P_e}{A}}{1+(n-1)p} \ldots\ldots\ldots\ldots\ldots (13)$$

Now, the *total unit steel stress* is the sum of f_{si} (eq. 7) due to the *internal* load, plus the unit steel stress due to the *second* part of the external load *only*, because during the application of the first part of the external load the steel has been neutral. Substituting the second part of the external load (eq. 10) for $\frac{P}{A}$ in eq. 3, we have the unit steel stress due to the external load.

$$f_{se} = \frac{n}{1+(n-1)p}\left[\frac{P_e}{A} - \frac{E}{n}m(1-p)\right] \ldots\ldots\ldots\ldots (14)$$

Therefore, *the total unit steel stress*

$$f_s = f_{si} + f_{se}$$

Adding equations 7 and 14, and reducing, we have

$$f_s = \frac{n}{1+(n-1)p} \times \frac{P_e}{A} \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

Eqs. 13 and 15 are the original general equations 2 and 3, with $\frac{P}{A}$ replaced by $\frac{P_e}{A}$.* THIS MEANS JUST THIS:

Although there is an internal load (eq. 5) due to the potential anti-contractive stress, which load puts a compressive stress f_{si} (eq. 7) into the steel and reduces the potential anti-contractive tensile stress in the concrete (eq. 4) to the residual tensile stress f_{cr} (eq. 8), subject to certain restrictions stated later on, *the strength of the column for carrying its external load is just the same as it would be if there were no shrinkage of the concrete and no underlying steel compressive stress f_{si} and concrete (residual) tensile stress f_{cr}.*

Some of the compressive strength of the *steel* has been used up in helping to sustain the internal load, but the compressive strength of the *concrete* has *not* been impaired by the compressive stress f_{ci} by the internal load, said compressive stress has only reduced the anti-contractive *tensile* stress f_h to the smaller residual tensile stress f_{cr}, and the concrete due to being in this latter state of tensile stress has a greater capacity for absorbing pressure coming upon it by the external load than it would have in a state of zero stress. And not until this *tensile* stress in the concrete has, by the pressure upon it by the external load, been converted into the *compressive* stress $= \frac{1}{n}$ of that of the steel, can the latter begin to act with the concrete in sustaining the external load. Eq. 13 and 15 show conclusively that the compressive strength of the steel given up to the *internal* load is conserved in the greater capacity for pressure of the concrete, the latter, by physical law, bears *all* the external load *up to* the attainment of elastic equality with the steel.

With respect to the external load, *the "impaired" strength of the steel is offset by the "improved" strength of the concrete to a precise balance.* The final *resultant* concrete and steel stresses, f_c and f_s, are not affected by the concrete shrinkage *at all, provided the external load is at least as large as* $\frac{E}{n} m(1-p)$.

The designers who have made use of eq. 1 with the understanding that P in said equation is nothing else but the external load (not being aware of the other load in action) have *generally* been correct in doing so without, maybe, being quite sure of it.

RETROSPECT

In presenting the development beginning after eq. 8, it has been silently assumed that the external load $\frac{P_e}{A}$ is large enough to *not only* furnish $f_{c1} = -f_h = \frac{E}{n} \times m$, the *exclusive unit pressure on the* concrete by the compensating load $\frac{E}{n} \times m(1-p)$, but also f_{c2}, an additional *apportioned* pressure on the

* The very obvious reason for the resulting equations 13 and 15 is concisely stated later on in Commentary on Table "B."

concrete by the *second* part, $\frac{P_e}{A} - \frac{E}{n} \times m(1-p)$ of the *whole external* load $\frac{P_e}{A}$. In some cases, however, this assumption does *not* apply, e.g., in our numerical example the compensating load $1000 \times (1-p)$ would be greater than the external load $\frac{P_e}{A} = 750 \times (1+14p)$ for values of p <0.0217, and in these cases $750 < f_{cr} + \frac{E}{} \times m$, or $< f_{cr} + f_{c1}$ or $< f_{cr} + 1000$ (see Diagram 4), there is *less* external load than required to make the resultant concrete compressive stress $= \frac{1}{n}$ of the compressive steel stress f_{si}, and there is *no* second part at all of the external load to furnish any stress f_{c2}, (or f_{se} either). At the time when elastic equilibrium is *just* attained, we have $f_{c2} = 0$ and eq. 12 becomes

$$f_c = f_{cr} + f_{c1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12^a)$$

Substituting values from eq. 8 and 9, and reducing

$$f_c = \frac{\frac{E}{n} m(1-p)}{1 + (n-1)p} = f_{ci} \quad \ldots\ldots\ldots\ldots (12^b)$$

or

$$f_c \left[1 + (n-1)p\right] = \frac{E}{n} m\ (1-p) \quad \ldots\ldots (12^c)$$

With the special values of $f_c = 750$, $n = 15$, $m = 0.0005$, $\frac{E}{n} = 2{,}000{,}000$ substituted into eq. 12^c, we have

$$750\ (1 + 14p) = 1000\ (1-p)$$

Solving for p, we have $p_{min} = 0.02174$, for this special case. Using eq. 12^c in its general form, with the exception of introducing $n = 15$, and, therefore, $\frac{E}{n} = 2{,}000{,}000$, and solving for p, we have

$$p_{min} = \frac{2{,}000{,}000 \times m - f_c}{2{,}000{,}000 \times m + 14 f_c} \quad \ldots\ldots\ldots\ldots\ldots\ldots (12^d)$$

These (minimum) values of p—for 4 different values of f_c and 5 different values of m—are assembled in Table I below.

TABLE I—MINIMUM VALUES OF p FOR ATTAINMENT OF ELASTIC EQUILIBRIUM:

$n = 15$

f_c	m				
	0.0004	0.00045	0.0005	0.00055	0.0006
650#	0.0152	0.0246	0.0347	0.0444	0.0534
700#	0.0093	0.0186	0.0278	0.0367	0.0455
750#	0.0048	0.0140	0.0217	0.0305	0.0384
800#	0.0000	0.0084	0.0164	0.0244	0.0323

These minimum p values for any one m decrease when f_c increases.
These minimum p values for any one f_c increase when m increases.

For any one *given* f_c together with any one *accepted* m (the unrestrained unit contraction of the concrete)—the table shows at once the minimum steel ratio p which will permit us to use eq. 1, construing the P therein as the *actual external* load on the column. In a column designed with a $p \geqq$ the tabular value—for a *given* f_c and an *accepted* m—the resultant concrete and steel stresses are *not* affected by the shrinkage of the concrete. On the other hand, the load on a column designed with a steel ratio $p <$ the tabular value —for any *given* f_c and any *accepted* m—is *not* represented by eq. 1. Taking a special *example:* the *given* $f_c = 750\#$, the *accepted* $m = 0.0005$, $p = 0.01$, this is less than the required minimum p $(= 0.0217)$ for elastic equilibrium: Elastic equilibrium *not* being attained, all the external load rests on the concrete *only*.

We have

$$f_c = f_{cr} + f'_{c1}*$$

where f'_{c1} denotes the unit pressure upon the concrete, due to the full external load $\frac{P_e}{A}$, on the unit size (1 sq. in.) reinforced concrete column, this unit pressure $= \frac{\frac{P_e}{A}}{1-p}$ in the given cases $= \frac{\frac{P_e}{A}}{0.99}$. By eq. 8,

$$f_{cr} = -\frac{0.01}{1.14} \times 15{,}000 = -132.$$

Therefore, we have $f'_{c1} = f_c - f_{cr} = 750 + 132 = 882$ lb., and the *external* load on the unit size (1 sq. in.) reinforced concrete column

$$\frac{P_e}{A} = 882 \times 0.99 = 873 \text{ lb.}$$

The steel stress is that due to the internal load $\frac{E}{n} m\,(1-p)$, it is by eq. 7

$$f_{si} = \frac{0.99}{1.14} \times 15{,}000 = 13{,}030 \text{ lb.}$$

Were we to assume $m = 0.0006$, we would have $f_{cr} = -\frac{0.01}{1.14} \times 18{,}000 = -158$

$$\text{and } f'_{c1} = 750 + 158 = 908 \text{ lb.}$$

and the *external* load on the unit size (1 sq. in.) column $= 908 \times 0.99 = 899$ lb. and the steel stress $f_{si} = \frac{0.99}{1.14} \times 18{,}000 = 15{,}630$ lb.

Eq. 1, ignoring the internal load and the anti-contractive stress, as it *does*, would give us, in either case $\frac{P}{A} = 750 \times 1.14 = 855$ lb. as the external column

* Accented terms, such as f'_{c1}, $\frac{P'_e}{A}$, f'_{cr}, f_s''s denote less than the final or full respective terms f_c, $\frac{P_e}{A}$, f_{cr}, f_{se}.

load, which, to be sure, is less than the load when the resultant concrete stress = 750 lb., but the equation would also tell us that the steel stress = 750 × 15 = 11,250 lb., while it is 13,030 with $m = 0.0005$ and 15,630 with $m = 0.0006$. [A steel stress of 11,250 lb. will be realized if the unrestrained unit contraction were as small as 0.00043.] It is not within our control to reduce the steel stress f_{si}, because it is exclusively a result of the *internal* load $\frac{E}{n} \times m\,(1-p)$, and a reduction of the *external* load to less than 873 or 899, or 855, respectively, will not take an iota from the stress f_{si} (= 13,030 or 15,630, resp.),—it is opportune to state here: the steel stress determines the deformation of the column, therefore we may expect a greater deformation in columns of $p < 0.02175$ than in columns of a $p > 0.02175$, because the steel stress in the former case is f_{si} by eq. 7, and this stress is greater than f_s by eq. 3 or 15 in the latter case.

TABLE A

	30 DAYS	90 DAYS	180 DAYS
	COMPR. STRENGTH = $\frac{f}{8}$ (62½%)	COMPR. STRENGTH = $\frac{f}{10}$ (60%)	COMPR. STRENGTH = f (100%)
UNRESTRAINED CONTRACTION	$m' = \frac{f}{E} = m = .0003125$	$m'' = \frac{f}{E} = m = .0004$	$m = .0005$
n OF CONCRETE 30, 90 AND 180 DAYS, RESP.	$n' = \frac{E}{E_c} = 12 = 24$	$n'' = \frac{E}{E_c} = n = 18\frac{2}{3}$	$n = 15$
MODULUS OF ELASTICITY CONCRETE $\frac{E}{n'} = \frac{E_c}{24}$ AND $\frac{E}{n} = \frac{E_c}{15}$, RESP.	$\frac{E}{n'} = \frac{E}{24} = \frac{15}{24} \times \frac{E}{n} = 1250000$	$\frac{E}{n''} = \frac{E}{n} = \frac{15}{18\frac{2}{3}} \times \frac{E}{n} = 1,600,000$	$\frac{E}{n} = \frac{30,000,000}{15} = 2,000,000$
ANTI-CONTRACTIVE TENSION $f_s' = \frac{E}{24} \times \frac{f}{E} \times m' = \frac{f}{24} \times m' \times 1000$, ETC.	$f_s' = \frac{E}{24} \times \frac{f}{E} \times m' = (\frac{f}{8}) \times 1000 = 39^{\#}$	$f_s'' = \frac{E}{18\frac{2}{3}} \times \frac{f}{E} \times m = (\frac{f}{10}) \times 1000 = 640^{\#}$	$f_s = \frac{E}{n} \times m = 2,000,000 \times .0005 = 1000^{\#}$
CONSERVATIVE ULTIMATE TENSILE STRENGTH	$\frac{f}{8} = 150 = 94^{\#}$	$\frac{f}{10} = 150 = 120^{\#}$	$= 150^{\#}$
f_{c1}', f_{c1}'' AND f_{c1} PRESSURES (FROM CONCRETE) BY INTERNAL LOAD, $p = .01$	$\frac{144}{144}(\frac{f}{8})^2 = 867 = 394^{\#} = f_{c1}'$	$\frac{144}{144}(\frac{f}{10})^2 = 867 = 539^{\#} = f_{c1}''$	$867^{\#} = f_{c1} = \frac{1-p}{1-0.02175p} \times \frac{f}{8} \times m$ (Eq 6)
$p = .02$	$\frac{138}{144} = " = 765 = 262^{\#}$	$\frac{138}{144} = " = 765 = 470^{\#}$	$765^{\#}$
$p = .03$	$\frac{132}{144} = " = 680 = 224^{\#}$	$\frac{132}{144} = " = 680 = 402^{\#}$	$680^{\#}$
$p = .04$	$\frac{126}{144} = " = 615 = 196^{\#}$	$\frac{126}{144} = " = 615 = 359^{\#}$	$615^{\#}$
$p = .05$	$\frac{120}{144} = " = 557 = 172^{\#}$	$\frac{120}{144} = " = 557 = 321^{\#}$	$557^{\#}$
$p = .06$	$\frac{114}{144} = " = 510 = 154^{\#}$	$\frac{114}{144} = " = 510 = 291^{\#}$	$510^{\#}$
$p = .07$	$\frac{108}{144} = " = 470 = 139^{\#}$	$\frac{108}{144} = " = 470 = 266^{\#}$	$470^{\#}$
$p = .08$	$\frac{102}{144} = " = 433 = 126^{\#}$	$\frac{102}{144} = " = 433 = 242^{\#}$	$433^{\#}$
$p = .09$	$\frac{96}{144} = " = 402 = 116^{\#}$	$\frac{96}{144} = " = 402 = 226^{\#}$	$402^{\#}$
$p = .10$	$\frac{90}{144} = " = 376 = 106^{\#}$	$\frac{90}{144} = " = 376 = 207^{\#}$	$374^{\#}$
(30 DAYS) $f_{c1}' = f_s' + f_{c1}'$ (90 DAYS) $f_{c1}'' = f_s'' + f_{c1}''$ (180 DAYS) $f_{c1} = f_s + f_{c1}$ RESIDUAL TENSIONS, $p = .01$	$-391 + 394 = +3 = f_{c1}'$	$-640 + 539 = -101 = f_{c1}''$	$-135 = f_{c1} = f_s + f_{c1}$
$p = .02$	$" + 262 = -129$	$" + 470 = -170$	-235
$p = .03$	$" + 224 = -167$	$" + 402 = -238$	-320
$p = .04$	$" + 196 = -195$	$" + 359 = -281$	-385
$p = .05$	$" + 172 = -219$	$" + 321 = -319$	-443
$p = .06$	$" + 154 = -237$	$" + 291 = -349$	-488
$p = .07$	$" + 139 = -252$	$" + 266 = -374$	-530
$p = .08$	$" + 126 = -265$	$" + 242 = -398$	-567
$p = .09$	$" + 116 = -275$	$" + 226 = -416$	-598
$p = .10$	$" + 106 = -283$	$" + 207 = -433$	-626
TO DECREASE the RESIDUAL TENSIONS TO −94, −120, and −150, RESP.	P_{RES} on $\frac{D}{8}$	EXT. LOAD on $10''$ COL. $\frac{D}{8}$	P_{RES} on $\frac{D}{10}$	EXT. LOAD on $10''$ COL. $\frac{D}{10}$	P_{RES} on $\frac{D}{8}$	EXT. LOAD on $10''$ COL. $\frac{D}{8}$
1. REQUIRES PRESSURES per $\square''$ of CONCRETE	$p = .01$					
2. CORRESPONDING LOAD per $\square''$ COL.	$p = .02$	$+35^{\#} = 98 = 34^{\#} = 3.54\%$ of $\frac{D}{A}$	$+50^{\#} = 98 = 49^{\#} = 5.1\%$ of $\frac{D}{A}$	$+85^{\#} = 98 = 83^{\#} = 8.65\%$ of $\frac{D}{A}$
3. THE LATTER in % of $\frac{D}{A}$.	$p = .03$	$+73^{\#} = 91 = 71^{\#} = 6.66$	$+110^{\#} = 97 = 106^{\#} = 10.7$	$+170^{\#} = 97 = 165^{\#} = 15.5$
	$p = .04$	$+101^{\#} = 96 = 97^{\#} = 9.3$	$+161^{\#} = 96 = 155^{\#} = 13.25$	$+235^{\#} = 96 = 225^{\#} = 19.25$
	$p = .05$	$+125^{\#} = 96 = 119^{\#} = 9.35$	$+199^{\#} = 95 = 189^{\#} = 14.65$	$+293^{\#} = 95 = 278^{\#} = 21.8$
	$p = .06$	$+143^{\#} = 94 = 134^{\#} = 9.7$	$+229^{\#} = 94 = 215^{\#} = 15.6$	$+330^{\#} = 94 = 310^{\#} = 23.0$
	$p = .07$	$+158^{\#} = 93 = 147^{\#} = 9.9$	$+254^{\#} = 93 = 236^{\#} = 15.9$	$+380^{\#} = 93 = 353^{\#} = 23.8$
	$p = .08$	$+171^{\#} = 92 = 157^{\#} = 9.86$	$+276^{\#} = 92 = 256^{\#} = 16.1$	$+417^{\#} = 92 = 389^{\#} = 24.1$
	$p = .09$	$+181^{\#} = 91 = 165^{\#} = 9.74$	$+296^{\#} = 91 = 269^{\#} = 15.9$	$+446^{\#} = 91 = 407^{\#} = 24.0$
	$p = .10$	$+191^{\#} = 90 = 172^{\#} = 9.55$	$+313^{\#} = 90 = 282^{\#} = 15.65$	$+476^{\#} = 90 = 430^{\#} = 23.9$
(THESE FIGURES APPEAR in the ABOVE ORDER)				

→ THESE PERCENTAGES ARE THE ORDINATES OF FIG. 1 or DIAGRAM 8

FOR IMMEDIATE PRACTICAL USE, HOWEVER, THE GRAPHS OF FIGS. 2, 3, 4, 5 and 6 HAVE BEEN PLOTTED, THEIR ORDINATES HAVE BEEN DEDUCED BY MULTIPLYING THOSE OF FIG. 1 BY THE COEFFICIENTS $\frac{D+L}{D}$

	FIG. 2	FIG. 3	FIG. 4	FIG. 5	FIG. 6
LIVE LOAD per $\square'' = L$	$60^{\#}$	$100^{\#}$	$200^{\#}$	$300^{\#}$	$350^{\#}$
DEAD " " " $= D$	$75^{\#}$	$100^{\#}$	$111^{\#}$	$137^{\#}$	$146^{\#}$
COEFFICIENTS $\frac{D+L}{D}$	$\frac{135}{75} = 1.8$	$\frac{200}{100} = 2.0$	$\frac{311}{111} = 2.8$	$\frac{437}{137} = 3.2$	$\frac{496}{146} = 3.4$

In all cases, when $p \geqq$ the values of Table I, the resultant steel stress f_s is n times the permitted concrete stress (when $f_c = 750$ $f_s = 15 \times 750 = 11{,}250$ lb.). The p values below the heavy zig-zag line of Table I are *barred* by the Joint Committee Specifications. To all columns with $p <$ the tabulated values—for any *given* f_c and *adopted* m—the statement "the impaired strength of the steel is offset by the improved strength of the concrete to a precise balance" does *not* apply. Table I indicates that the lower limit of the steel ratio (Joint Committee Specifications) should be *raised* for the purpose of attaining elastic equilibrium, as well as having lower f_{si} stresses (see Diagram 3), and to have eq. 1 give the correct load for the working stress f_c.

The diagrams show evidence of some of the above conditions, viz:—

1. The $\frac{P_e}{A}$ graph on Diagram 2 shows a break at $p = 0.02174$.

2. If n were $= 20$ (which is not to be expected except under a later "plastic flow"), eq. 1 would hold for all steel ratios $p -$ see Diagram 4.

3. Diagram 1, first figure, and Diagram 2 show the compensating load graphs intersecting the $\frac{P_e}{A} + \frac{P_i}{A}$ and the $\frac{P_e}{A}$ graphs, respectively, at $p = 0.02174$.

Comment on Diagram 7.—Shows for any $\frac{P'_e}{A}$ from $\frac{P'_e}{A} = 0$ up to $= \frac{P_e}{A} = 750\,[1+(n-1)p]$ the corresponding *resultant* unit concrete and steel stresses—for columns with steel ratios from $p = 0.01$ to $p = 0.08$.

The diagram is self-explanatory, the resultant unit concrete stresses are

$$f_c = \frac{\frac{P'_e}{A}}{1-p} + f_{cr} \quad \text{when } \frac{P'_e}{A} < \frac{E}{n}\, m\,(1-p)$$

$$\text{and} \quad f_c = \frac{\frac{P'_e}{A}}{1+(n-1)p} \text{ when } \frac{P'_e}{A} > \frac{E}{n}\, m\,(1-p)$$

$$(\text{and} \leqq \frac{P_e}{A} \text{ (the design load)})$$

These resultant unit concrete stresses are tensile stresses when the $\frac{P'_e}{A}$ is relatively small, as the diagram clearly shows.

The resultant unit concrete stress approaches zero when $\frac{P'_e}{A}$ in a

1% (steel)	column	approaches	131 lb.	=	15%	of its	design	load	$-\frac{P_e}{A}$	= 855
2%	"	"	229	"	= 24%	"	"	"	"	= 960
3%	"	"	307	"	= 29%	"	"	"	"	= 1065
4%	"	"	369	"	= 32%	"	"	"	"	= 1170
5%	"	"	422	"	= 33%	"	"	"	"	= 1275
6%	"	"	459	"	= 33%	"	"	"	"	= 1380
7%	"	"	492	"	= 33%	"	"	"	"	= 1485
8%	"	"	520	"	= 33%	"	"	"	"	= 1590

Assuming 150 lb. as a conservative ultimate tensile strength of a 1:2:4 concrete (the ordinate of this tensile stress is indicated in the diagram), we see that by the time when hardening is practically complete and the full residual tension would be developed, the applied loads $\frac{P'_e}{A}$ should be

$\geqq$	0 in a	1%	(steel)	column	$\geqq$	0%	of its	design	load	$\frac{P_e}{A}$ = 855
$\geqq$	82 " "	2%	"	"	$\geqq$	9%	"	"	"	= 960
$\geqq$	161 " "	3%	"	"	$\geqq$	15%	"	"	"	= 1065
$\geqq$	225 " "	4%	"	"	$\geqq$	19%	"	"	"	= 1170
$\geqq$	276 " "	5%	"	"	$\geqq$	22%	"	"	"	= 1275
$\geqq$	318 " "	6%	"	"	$\geqq$	23%	"	"	"	= 1380
$\geqq$	352 " "	7%	"	"	$\geqq$	24%	"	"	"	= 1485
$\geqq$	382 " "	8%	"	"	$\geqq$	24%	"	"	"	= 1590

A more extended study of this problem has been made and is presented in Appendix II—"A Study of the Steel Ratio p of the Column."
The resultant unit steel stresses are

$$f_s = f_{si} = \frac{1-p}{1+(n-1)p} Em, \text{ when } \frac{P'_e}{A} < \frac{E}{n} m\,(1-p)$$

$$f_s = \frac{n}{1+(n-1)p} \times \frac{P'_e}{A}, \text{ when } \frac{P'_e}{A} > \frac{E}{n} m(1-p) \left(\text{and} \leqq \frac{P_e}{A}\right)$$

Commentary on Table "B"

Concrete and Steel Stresses are tabulated for three (3) reinforced concrete columns, viz., for columns of $p = 0.01$, $p = 0.04$, and $p = 0.06$, respectively.

(1) The stresses in column A are those derived from eq. 2 and 3. As these equations do not take any account of shrinkage stresses, the stresses computed from them are necessarily the concrete and steel stresses of columns having an imaginary "non-shrinking" concrete. These stresses are purely hypothetical as long as $\frac{P'_e}{A} < \frac{E}{n} m(1-p)$. However, when $\frac{P'_e}{A} \geqq \frac{E}{n} m(1-p)$, equations 2 and 3 give the correct "real" stresses for columns with a "real" shrinking concrete, *because* when $\frac{P'_e}{A} = \frac{E}{n} m(1-p)$ the resultant concrete

stress is $f_{cr} - f_h = f_{ci}$ (see eq. 8, 4 and 6, or Diagram 4), and the *steel* stress is f_{si} (eq. 7), f_{ci} and f_{si} are "apportioned" stresses obtained by substituting the internal load $\frac{E}{n}\, m(1-p)$ for $\frac{P}{A}$ in eq. 2 and 3. The additive *concrete* stress f'_{c2} (eq. 11) and the additive *steel* stress f'_{se} (eq. 14) for the excess of the external load $\frac{P'_e}{A}$ over the compensating load $\frac{E}{n}\, m(1-p)$—in other words: the *second* part of the external load,—are both also "apportioned" stresses obtained by substituting the said second part of the external load for $\frac{P}{A}$ in eq. 2 and 3. It follows then that the sums of f_{ci} and f'_{c2}, as well as the sums of f_{si} and f'_{se}, as such, are also "apportioned" stresses, $f_{ci} + f'_{c2}$ is the f_c of eq. 2, $f_{si} + f'_{se}$ is the f_s of eq. 3. Therefore, the stresses in column A—for external load $\geqq \frac{E}{n}\, m(1-p)$—are identical with the corresponding "Real" Resultant Stresses "C" of the table.

TABLE B

THE "REAL" CONCRETE AND STEEL STRESSES

$$\left.\begin{aligned} f'_c &= \frac{\frac{P'_e}{A}}{1-p} + f'_{cr} \\ f'_s &= f'_{si} \end{aligned}\right\} \text{FOR EXTERNAL LOADS } \frac{P'_e}{A} \geqq \frac{E}{n}\, m(1-p)$$

$$\left.\begin{aligned} f'_c &= \frac{\frac{P'_e}{A}}{1+(n-1)p} \\ f'_s &= n f'_c \end{aligned}\right\} \text{FOR EXTERNAL LOADS } \frac{P'_e}{A} \geqq \frac{E}{n}\, m(1-p) \;\; T_0 \geqq \frac{P_e}{A}$$

"DESIGN" LOADS [OR FRACTIONS OF SAME] ARE UNDERSTOOD TO BE THE "EXTERNAL" LOADS

"DESIGN" LOAD $\frac{P_e}{A} = f'_c\,[1+(n-1)p]$

$f'_c = 750$, $n = 15$

$E = 30{,}000{,}000$ $m = 0.0005$ $n = 15$

THE "REAL" RESULTANT STRESSES

STEEL RATIO	EXTERNAL LOAD INCREASING BY $\frac{P'_e}{A}$ OR $\frac{P'}{A}$	$f'_c = \frac{\frac{P'_e}{A}}{1+(n-1)p}$ (Eq. 2) $f'_s = n f'_c$ (Eq. 3) HYPOTHETICAL STRESSES FOR "NEW" SUPERIMPOSED "TIME" $\frac{P'_e}{A}$	[FAULTY] COMBINATION OF THE "HYPOTHETICAL" EXTERNAL LOAD STRESSES WITH THE "REAL" STRESSES DUE TO SHRINKAGE OF CONCRETE		CORRECT COMBINATION OF THE UNIT PRESSURES BY THE EXTERNAL LOADS WITH THE "REAL" STRESSES DUE TO SHRINKAGE OF THE CONCRETE		NOTES
		UNIT STRESSES	UNIT STRESSES		UNIT STRESSES		
		CONCRETE STEEL	CONCRETE	STEEL	CONCRETE	STEEL	
$p=.01$	$\frac{P'_e}{A}=0$ = 85.5	75 1125	−132 − 75 = − 57	13030 + 1125 = 14155	−132 − 86 = − 46	13030	$\frac{P_e}{A} = f'_c[1+(n-1)p] = 750(1+14\times.01) = 750\times1.14 = 855$# "DESIGN" LOAD
	" = " = 171	150 2250	" + 150 = + 18	" + 2250 = 15280	" + 175 = + 61	"	$f_{cr} = \frac{P}{1+(n-1)p} \times Em = -\frac{61}{1.14} \times 30{,}000{,}000 \times .0005 = -133$#
	" = " = 256.5	225 3375	" + 225 = + 93	" + 3375 = 16450	" + 275 = + 127	"	$f_{si} = \frac{nP}{1+(n-1)p} \times Em = \frac{99}{1.14} \times 30{,}000{,}000 \times .0005 = 13030$#
	" = " = 342	300 4500	" + 300 = + 168	" + 4500 = 17330	" + 395 = + 213	"	
	" = " = 427.5	375 5625	" + 375 = + 243	" + 5625 = 18655	" + 435 = + 300	"	
	" = " = 513	450 6750	" + 450 = + 318	" + 6750 = 19780	" + 548 = + 386	"	
	" = " = 598.5	525 7875	" + 525 = + 393	" + 7875 = 20905	" + 608 = + 472	"	
	" = " = 684	600 9000	" + 600 = + 468	" + 9000 = 22030	" + 691 = + 559	"	COMPENSATING LOAD. $\frac{E}{n}\, m(1-p)$ SHOULD BE $\frac{30{,}000{,}000}{15} \times .0006 \times .99 = 1188$#
	" = " = 769.5	675 10125	" + 675 = + 543	" + 10125 = 23155	" + 777 = + 645	"	NOTE. THE "DESIGN" LOAD (855) IS < COMPENSATING LOAD
	" = " = 855	750 11250	" + 750 = + 618	" + 11250 = 24280	" + 862 = + 700	"	
$p=.04$	$\frac{P'_e}{A}=0$ = 117	75 1125	− 394 − 75 = − 319	9220 + 1125 = 10345	− 394 − 122 = − 262	9220	$\frac{P_e}{A} = 750 \times 1.56 = 1170$#, "DESIGN" LOAD
	" = " = 234	150 2250	" + 150 = − 244	" + 2250 = 11470	" + 244 = − 150	"	$f_{cr} = -\frac{94}{1.56} \times 15000 = -3944$#
	" = " = 351	225 3375	" + 225 = − 169	" + 3375 = 12595	" + 344 = − 18	"	$f_{si} = \frac{96}{1.56} \times 15000 = 9220$#
	" = " = 468	300 4500	" + 300 = − 94	" + 4500 = 13720	" + 488 = + 194	"	
	" = " = 585	375 5625	" + 375 = − 9	" + 5625 = 14845	" + 610 = + 326	"	COMPENSATING LOAD = 1000 × .96 = 960#
	" = " = 702	450 6750	" + 450 = + 66	" + 6750 = 15970	" + 732 = + 348	"	
	" = " = 819	525 7875	" + 525 = + 141	" + 7875 = 17095	" + 854 = + 470	"	
	" = " = 936	600 9000	" + 600 = + 216	" + 9000 = 18220	" + 976 = + 592	"	
	$(1-p)$ = 960	6754 10125+	" + 675 = + 231	" + 9225 = 18445	" + 615	"	
	1170 = 1053	675+ 10125+	" + 675 = + 291	" + 10125 = 19345	" + 675	10125	
	" = " = 1170	750+ 11250+	" + 750 = + 366	" + 11250 = 20470	" + 750	11250	
$p=.06$	1380 = 738	75 1125	− 488 − 75 = − 413	7630 + 1125 = 8775	488 + 147 = 341	7630	$\frac{P_e}{A} = 750 \times 1.84 = 1380$#, "DESIGN" LOAD
	" = " = 276	150 2250	" + 150 = − 338	" + 2250 = 9900	" + 294 = − 194	"	$f_{cr} = -\frac{294}{1.84} \times 15000 = -4888$#
	" = " = 414	225 3375	" + 225 = − 263	" + 3375 = 11025	" + 441 = − 68	"	$f_{si} = \frac{294}{1.84} \times 15000 = 7650$#
	" = " = 552	300 4500	" + 300 = − 188	" + 4500 = 12150	" + 588 = + 79	"	
	" = " = 690	375 5625	" + 375 = − 113	" + 5625 = 13275	" + 734 = + 246	"	COMPENSATING LOAD = 1000 × .94 = 940#
	" = " = 828	450 6750	" + 450 = − 38	" + 6750 = 14400	" + 881 = + 393	"	
	$m(1-p)$ = 940	5104 7650+	" + 510 = + 22	" + 7650 = 15300	+ 510	7875	
	1380 = 966	525+ 7876+	" + 525 = + 37	" + 7875 = 15525	+ 525	7875	
	" = " = 1104	6004 9000+	" + 600 = + 112	" + 9000 = 16650	+ 600	9000	
	" = " = 1242	675+ 10125+	" + 675 = + 187	" + 10125 = 17775	+ 675	10125	
	" = " = 1380	750+ 11250+	" + 750 = + 262	" + 11250 = 18900	+ 750	11250	
		A	B		C		

READ COMMENTARY: CONCRETE AND STEEL STRESSES IN COLUMN "A" FOR $\frac{P'_e}{A} \geqq \frac{E}{n}\, m(1-p)$ — ARE "REAL" THEY ARE IDENTICAL WITH THE STRESSES IN THE "C" COLUMN. FOR EXPLANATION OF STRESSES IN "B" COLUMN " " " " " "C" "

(2) The "Real" Resultant *concrete* stresses in column C are

$$f_{cr} + \frac{\frac{P'_e}{A}}{1-p}, \text{ as long as } \frac{P'_e}{A} < \frac{E}{n} m(1-p)$$

and

$$f_{cr} + \frac{\frac{E}{n} m(1-p)}{1-p} + \frac{\frac{P'_e}{A} - \frac{E}{n} m(1-p)}{1 + (n-1)p}, \text{ when } \frac{P'_e}{A} \geqq \frac{E}{n} m(1-p)$$

The latter expression $\left[\text{for } \frac{P'_e}{A} \geqq \frac{E}{n} m(1-p)\right]$ reduces to

$$\frac{\frac{P'_e}{A}}{1 + (n-1)p}, \text{ the form of the right member of eq. 2.}$$

The "Real" *Steel* stresses in column C are

$$f_{si} \text{ (eq. 7), as long as } \frac{P'_e}{A} < \frac{E}{n} m(1-p)$$

and

$$f_{si} + f'_{se} = \frac{n}{1 + (n-1)p} \times \frac{P'_e}{A} \text{ (eq. 15), when } \frac{P'_e}{A} \geqq \frac{E}{n} m(1-p)$$

(All the stresses given in column C, as well as those for any other $\frac{P'_e}{A}$, or for any other p, may be read from Diagram 7.)

(3) The concrete stresses in column B are a combination of the residual tensile stresses of a "real" *shrinking* concrete with the compressive stresses (due to the external load) of an imaginary "non-shrinking" concrete.

The steel stresses, likewise, are a combination of the stresses due to the internal load of a "real" *shrinking* concrete with the compressive stresses (due to the external load) of an imaginary "*non-shrinking*" concrete. Both of these combinations, of course, are absurd.

The writer does not wish to be understood that these combinations have been made or have been suggested by any one, but, with the concrete *tensile* stress and the steel *compressive* stress, both due to the shrinkage of the concrete, now rationally determined by eq. 8 and 7 (and also in Appendix I), there might be an off-hand temptation suggesting such a combination.

In the foregoing analysis the anti-contractive stress f_h, the internal load $\frac{P_i}{A}$ due to same, the apportioned concrete and steel stresses f_{ci} and f_{si}, and the residual tension f_{cr}, have been developed in the given orders, before the external load was considered. It is not to be understood, however, that the effects of the anti-contractive stress and of the internal load are all completed before the external load combines its effects with the aforementioned. The stresses due to the resisted "hardening" contraction develop gradually and steadily (though not at a uniform rate) simultaneously with the pressures due to at least a part of the external load.

Looking at Diagram 4, the f_{cr} magnitude, (referring to the series of these values) shown there is its final value (completed hardening). While this value is getting its full growth, the external load pressure, at first the f_{c1} is also growing nearly simultaneously. The resultant of f_{cr} (negative) and $\frac{P'_e}{A}$ (positive), $\frac{P'_e}{A} < \frac{E}{n} \times m(1-p)$, will lie somewhere above f_{cr} (identical with the graph designated as f_{ci}) and below the "compensating" load graph, and we may feel assured that the said resultant point will not be so far below the 0 axis as to represent a tension greater than or as great as the tensile strength of the concrete. The development of f_{cr} towards its final magnitude is, however, independent of the progress of the construction, and if the latter should be rather slow, a tension in the concrete exceeding its tensile strength is conceivable. However, we will be optimistic and assume that enough external load will be piled up fast enough to insure the concrete's integrity during the precarious period.

$\overline{1-p}$

To show the joint action on the column by the two loads, internal and external, let us take this *example:*

A 4% steel, 1:2:4 concrete, column designed with $n=15$ and $f_s=750$ by eq. 1 gives $\frac{P}{A} = 1170$. To believe, however, that the concrete carries $(1-p) \times 750 = 0.96 \times 750 = 720$ and the steel carries $p \times n \times 750 = 0.04 \times 15 \times 750 = 450$

(1170)

of this (*external*) *load 1170*, as eq. 1 apparently tells us, would be an error.

By diagrams 3 and 4 we find

$$= 0.96 \times \left[1000 + \frac{1170-960}{1.56}\right]$$

the concrete carries $0.96 \times (f_{c1}+f_{c2})$ $= 0.96 \times (1000 + 135) =$

$= 0.96 \times 1135 = 1089$

the steel carries $0.04 \times f_{ss}$ $= 0.04 \times 2025 = 81$

(1170)

of this (external) load 1170 lb.

The internal load is 960 lb. (see diagram 1).

The concrete carries $0.96 \times f_{ci} = 0.96 \times 615$ lb. $= 591$ lb.

The steel carries $0.04 \times f_{si} = 0.04 \times 9220$ lb. $= 369$ lb. of this

Total $= 960$ lb. internal load.

Summing up, the concrete carries $1089 + 591 = 1680$

Summing up, the steel carries $81 + 369 = 450$

Total $= 2130$ lb. $(= 1170 + 960)$

The total pressure upon the 0.96 sq. in. of the concrete being 1680 lb., the unit pressure by the two loads is $\frac{1680}{0.96} = 1750$ lb. on a concrete which has a potential tensile stress (the anti-contractive tension) of 1000 lb., giving a resultant unit c ompressive stress of 750 lb., just as eq. 1 tell us, while the total

pressure upon the 0.04 sq. in. of steel being 450 lb. the unit pressure by the two loads is $\frac{450}{0.04} = 11{,}250$ lb., just as eq. 1 tell us, but both loads, the external as well as the internal load, and the anti-contractive tension are contributory to the resulting stresses, viz., to restate it

Concrete	1135 unit press. by external load	
	615 unit press. by internal load	
	−1000 anti-contr. tension	
Resultant =	750	
Steel		2025 unit press. by external load
		9220 unit press. by internal load
Resultant		11250

In the above example $p > 0.0217$ (see Table I), Eq. 13 and 15 with $f_c = 750$ and $f_s = 11{,}250$ give the value of $\frac{P_e}{A} = \frac{P}{A}$. Therefore, the $\frac{P}{A}$ and $\frac{P_e}{A}$ of eqs. 1, 1a, 2, 3, 9, 10, 11, 13, 14, 15 are interchangeable.

PLASTIC FLOW OF THE CONCRETE

Another question, viz: a *gradual increase of* n, the compressibility of the concrete with time, defined as "flow" by some and referred to as "time-yield" by Mr. Franklin R. McMillan in his paper "A Study of Column Test Data" in the 1921 Proceedings of the American Concrete Institute, is of *practical* interest to us, as it means a gradual increase in the steel stress over that based on $n = 15$ in eq. 1, by which the column strength was computed, and a gradual decrease of the concrete stress below that based on $n = 15$ in eq. 1.

It is a very simple matter to figure the steel and concrete stresses corresponding to any value of "n" other than 15. Equations 2 and 3 give the concrete and steel stress for any "n." The $\frac{P}{A}$ has been determined by a given f_c and a given "n" ($f_c = 750$ and $n = 15$ in our diagrams). This *fixes* the load on the column, and while "n" may change from its assumed initial value, $\frac{P}{A}$ is constant. Therefore, in the case of eq. 2, f_c varies directly with the value of $\frac{1}{1 + (n-1)p}$, and in the case of eq. 3, f_s varies directly with the value of $\frac{n}{1 + (n-1)p}$. The *decreasing* values of f_c and the *increasing* values of f_s for $n > 15$, viz., for $n = 30$, 40 and 50 are shown in Diagram 5 and will require no further comment. These f_c and f_s values check with the corresponding values of Diagrams 4 and 3 respectively, (f_s for $n = 50$ appearing complete in Diagram 5).

COMPARISON WITH MR. MCMILLAN'S DEVELOPMENT

Franklin R. McMillan in his aforementioned paper in the A.C.I. Proceedings states that "when the concrete stress has been brought to zero all the load (by which he means all the external load) *then* on the unit size (1 sq. in.) column is *Emp* (the steel stress *Em* multiplied with the steel area p)," but

the fact is: *All* the external load *then* on the column rests *on the concrete only* and is $Emp\ \frac{1-p}{1+(n-1)p}$. What Mr. McMillan says is *the external load on the column when concrete stress = o is* shown by the *Emp.* lines of Diagram 1. The *true external load at that time* is shown by the graphs of $Emp\ \frac{1-p}{1+(n-1)p}$, designated as "External Loads when concr. stress = *o*." The stress in the steel at *that* time is caused by the *internal load only*, it is (by eq. 7) $f_{si} = \frac{1-p}{1+(n-1)p} \times Em$, it is *not* due to *the external load or any part of* the *external load*. The external load as it increases to more than $Emp\ \frac{1-p}{1+(n-1)p}$ *continues to rest on the concrete only*, up to the time when the concrete stress has been increased to the compressive stress $\frac{1}{n} \times f_{si} = f_{ci}$. At that later time the concrete is just attaining its elastic equilibrium with the steel, and thenceforth *all further* (external) *load* is *apportioned* between the concrete and the steel *maintaining* this elastic equilibrium, in conformance with our eq. 1. The *external load*, at the time of elastic equilibrium, is $(1-p)\frac{E}{n}m$ (all on the concrete). The *whole external load* being $\frac{P_e}{A}$, it follows that only the *remaining* part of the external load, viz. $\frac{P_e}{A} - (1-p)\frac{E}{n}m$ is *apportioned* between the concrete and the steel, in conformance with our eq. 1.

Mr. McMillan's error is three-fold, in the first place, not *Emp* but $Emp\ \frac{1-p}{1+(n-1)p}$ is the external load resting upon the column when the concrete stress = *o*, and in the second place, not $\frac{P_e}{A} - Emp$ but $\frac{P_e}{A} - (1-p)\frac{E}{n}m$ is *the part of the external load apportioned* between the concrete and the steel, and furthermore, the change in the disposal of the external load, from *resting on the concrete only* to *being apportioned* between the concrete and the steel, occurs *neither* when the concrete stress is negative *nor* when it = *o*, but *when it is positive* and $= \frac{1}{n} f_{si} = f_{ci}$ (eq. 6).

Mr. McMillan's development led him to find a value of f_c (21st line, p. 165) *too small* by $\frac{Emp}{1+(n-1)p}$ (this writer's f_{cr} eq. 8), and a value of f_s (23d line) *too large* by $Em\ \frac{1-p}{1+(n-1)p}$ (this writer's f_{si} eq. 7).

Inasmuch as Mr. McMillan's results for f_c and f_s have been made the basis of the graded values of f_c given by eq. 43, p. 58, to be used in eq. 42 of the same page, of the *Joint Committee Report of 1924*, the values of $\frac{P_e}{A}$, corresponding to those f_c values for a 2000-lb. concrete, have been plotted in

Diagrams 1 and 2. The location of these points appears erratical in our graph, as we must expect. Consistent values of $\frac{P_e}{A}$ must lie on the line $\frac{P_e}{A}$ when $n = 15$, $f_c = 750$, or if another concrete working stress is used, on a line parallel to it, above or below as the case may be. The *Joint Committee values* f_c themselves (eq. 42) are plotted on Diagram 4, as far as they fall within the paper. These values f_c (for a 2000-lb. concrete) are 580, 660, 740, 820, 900, 980 for $p = 0.01$, 0.02, 0.03, 0.04, 0.05, 0.06, respectively (820, 900 and 980 are beyond the margin of the paper), but all of these *Joint Committee* f_c *values* appear on Diagram 5.

SUMMARY

Equations 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, also 1A, 9a, 12a, b, c, are perfectly general. Equation 12d, is special inasmuch as n has been taken = 15.

For the preparation of the seven (7) Diagrams and Tables A and B the physical constants have been assumed:

$E = 30{,}000{,}000$, $m = 0.0005$, and for the determination of the external load

$f_c = 750$ lb. and $n = 15$. Therefore, the external load in all diagrams is $\frac{P_e}{A} = 750\ (1 + 14p)$. For $\frac{P_e}{A}$, as we have shown, $\frac{P}{A}$ may be substituted, when $p \geqq 0.02174$ for $m = 0.0005$ [when $p < 0.02174$, $\frac{P_e}{A} = (f_c - f_{cr})\ (1-p)$.]

The value of E (the modulus of elasticity of steel) used is so generally accepted that no change of it will probably be suggested. The value of m, however, the unrestrained "hardening" contraction of concrete, may perhaps be questioned. A variation of m will affect f_h (eq. 4), $\frac{P_i}{A}$ (eq. 5), f_{ci} (eq. 6), f_{si} (eq. 7), f_{cr} (eq. 8), f_{c1} (eq. 9), f_{c2} (eq. 11), f_{se} (eq. 14). But f_c and f_s, the *resultant concrete and steel stresses* [always provided the external load $\frac{P_e}{A} \geqq \frac{E}{n} m\ (1-p)$] are not affected by m. The minimum value of p to which the balancing of *steel strength loss* with *concrete strength gain* does apply varies with m on the one hand, and with n and f_c, the former expressing the relative compressibility of the concrete and the latter the permissible working stress, on the other hand. An increase of m tends to increase, an increase in the "design" n and in the f_c tend to decrease the said p, and vice versa.

CONCLUSION

The foregoing development showing the anti-contractive tensile stress f_h creating an internal load on the column and, therefore, causing the stresses f_{si} and f_{ci}: due to said load, and furthermore the compounding of the anti-contractive stress f_h with the f_{ci} into the resultant f_{cr} (the residual tension)—has made it possible to *determine* the stresses in the two materials due to the "hardening" contraction of the concrete. The *opposite* nature and the *relative* magnitude of these stresses automatically divide the external load

into two parts, the first part being *absorbed by the concrete* excluding the steel from participation until elastic parity is established between the two materials, the remaining second part of the external load *apportioning* itself between the two materials. The problem of what effect the shrinkage of the concrete has on the strength of the column to support the visible (external) load *has thus been solved*, and the answer is nothing more than what intuition should have, or perhaps may have, told us.

SYNOPSIS OF STRESSES

I.—CASE OF $\frac{P_e}{A} \geqq \frac{E}{n} m(1-p)$ — OR: $p \geqq \frac{\frac{E}{n} m - f_c}{\frac{E}{n} m + (n-1) f_c}$

	Stresses	
	Concrete	Steel
Cause of Internal Load	$f_h = -\frac{E}{n} m$	
Effect of Internal Load	$f_{ct} = \frac{1-p}{1+(n-1)p} \times \frac{E}{n} m$	$f_{cr} = -\frac{p}{1+(n-1)p} \times Em$ — $f_{st} = \frac{1-p}{1+(n-1)p} \times Em$
Effect of External Load	$f_{c1} = -f_h = \frac{E}{n} m$	
	$f_{c2} = \frac{1}{1+(n-1)p}\left[\frac{P_e}{A} - \frac{E}{n} m(1-p)\right]$ Apportioned	$f_{se} = \frac{n}{1+(n-1)p}\left[\frac{P_e}{A} - \frac{E}{n} m(1-p)\right]$
Sum of Stresses (Resultants f_c and f_s)	$f_c = f_{cr} + f_{c1} + f_{c2} = \frac{\frac{P_e}{A}}{1+(n-1)p}$	$f_s = f_{st} + f_{se} = n \times \frac{\frac{P_e}{A}}{1+(n-1)p}$

NOTE: The P of Eqs. 2 and 3 may, therefore, be understood to be the "external" load P_e in this case.

II.—CASE OF $\frac{P_e}{A} < \frac{E}{n} m(1-p)$ — OR: $p < \frac{\frac{E}{n} m - f_c}{\frac{E}{n} m + (n-1) f_c}$

	Stresses	
	Concrete	Steel
Cause of Internal Load	$f_h = -\frac{E}{n} m$	
Effect of Internal Load	$f_{ct} = \frac{1-p}{1+(n-1)p} \times \frac{E}{n} m$	$f_{cr} = -\frac{p}{1+(n-1)p} \times Em$ — $f_{st} = \frac{1-p}{1+(n-1)p} E m$
Effect of External Load	$f'_{c1} = \frac{\frac{P_e}{A}}{1-p}$	
Sum of Stresses (Resultants f_c and f_s)	$f_c = f_{cr} + f'_{c1} = f_{cr} + \frac{\frac{P_e}{A}}{1-p}$	$f_s = f_{st} = \frac{1-p}{1+(n-1)p} \times Em$
	$\therefore \frac{P_e}{A} = (f_c - f_{cr})(1-p)$ the external load, for a given f_c.	

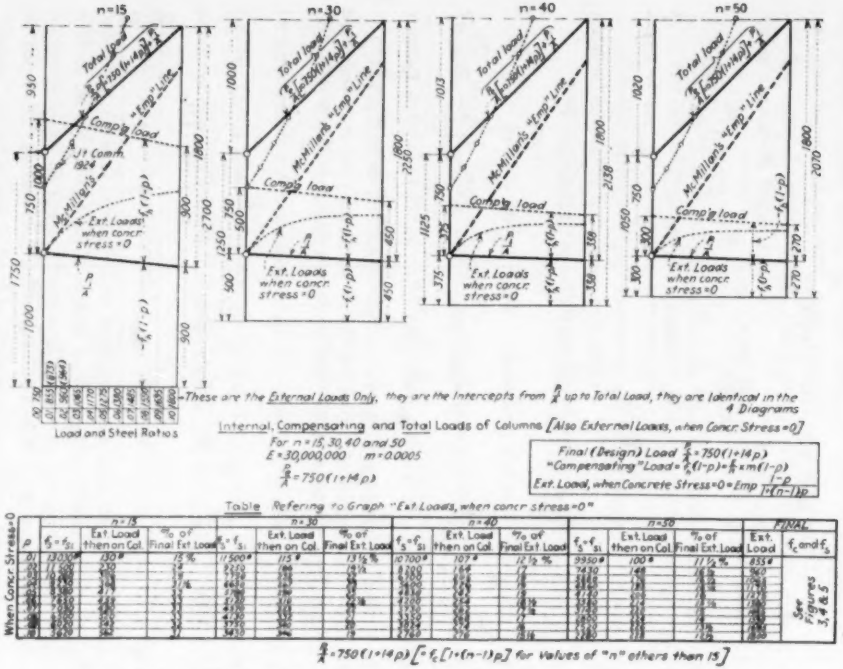


DIAGRAM 1.—Internal, Compensating and Total Loads (also External loads when concrete stress = O) for n = 15, 30, 40 and 50.

The total loads are $\frac{P_e}{A} + \frac{P_i}{A} = 750\ (1 + 14\ p) + \frac{E}{n}\ m\ (1-p)$

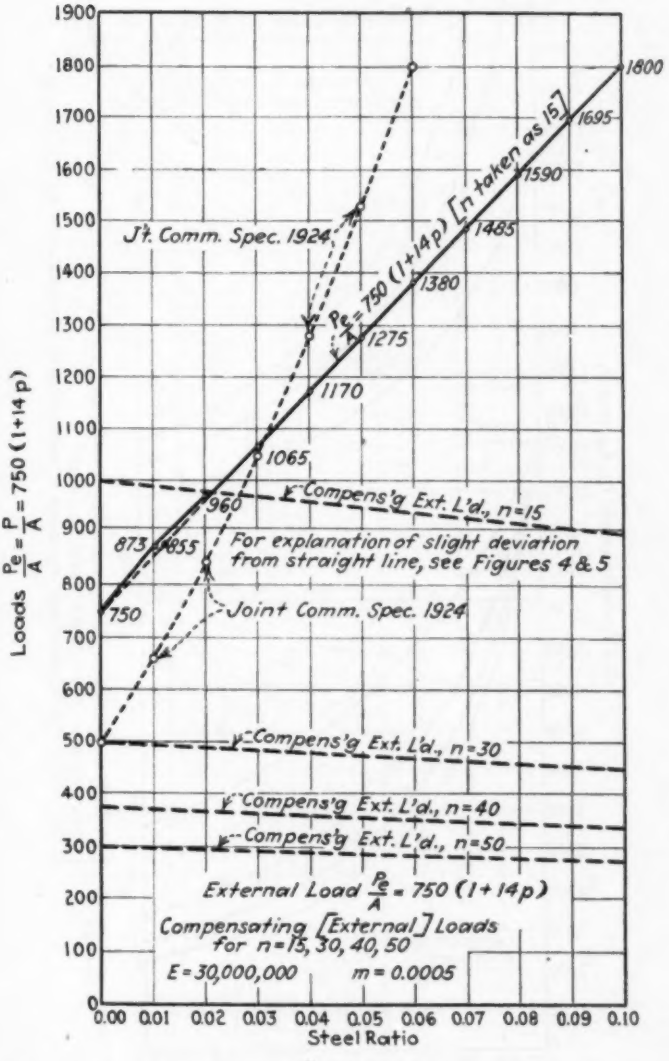


DIAGRAM 2.—External Loads $\frac{P_e}{A} = 750\ (1 + 14\ p)$ and Compensating External Loads $\frac{E}{n}\ m(1-p)$ for $n = 15, 30, 40$ and 50.

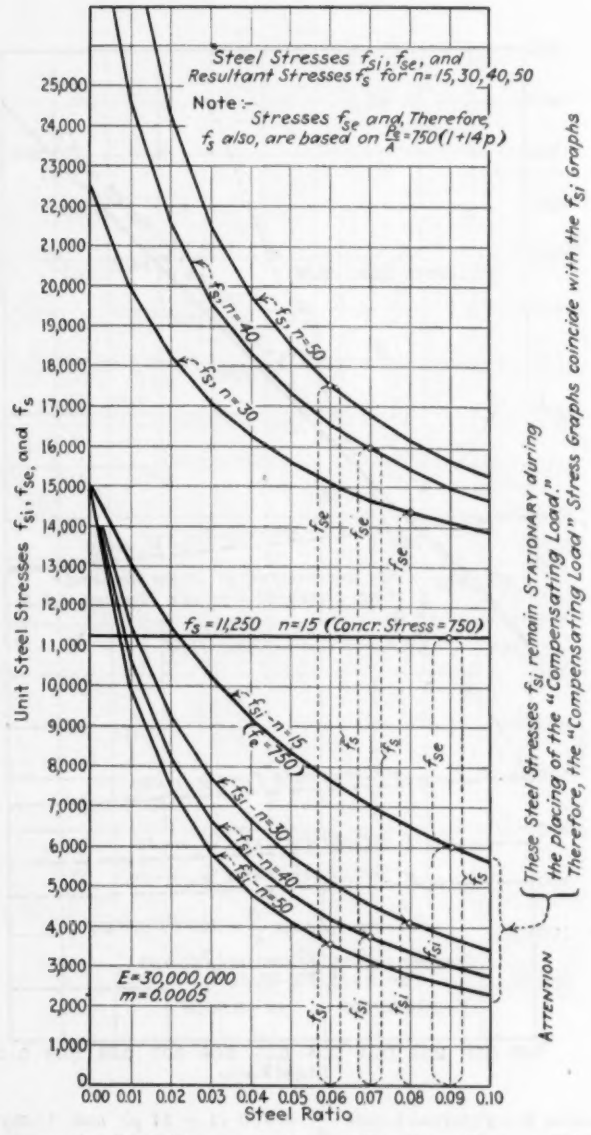

Steel Stresses f_{si}, f_{se}, and Resultant Stresses f_s for n=15, 30, 40, 50
Note:- Stresses f_{se} and, Therefore, f_s also, are based on $\frac{P_c}{A}=750(1+14p)$
Unit Steel Stresses f_{si}, f_{se}, and f_s
Steel Ratio
$f_s=11,250$ n=15 (Concr. Stress = 750)
E=30,000,000
m=0.0005
These Steel Stresses f_{si} remain STATIONARY during the placing of the "Compensating Load."
Therefore, the "Compensating Load" Stress Graphs coincide with the f_{si} Graphs
ATTENTION

DIAGRAM 3.—Steel Stresses f_{si}, f_{se} and Resultant Stresses f_s for $n = 15$, 30, 40 and 50.

[$f_s = 11250$ for $n = 15$]

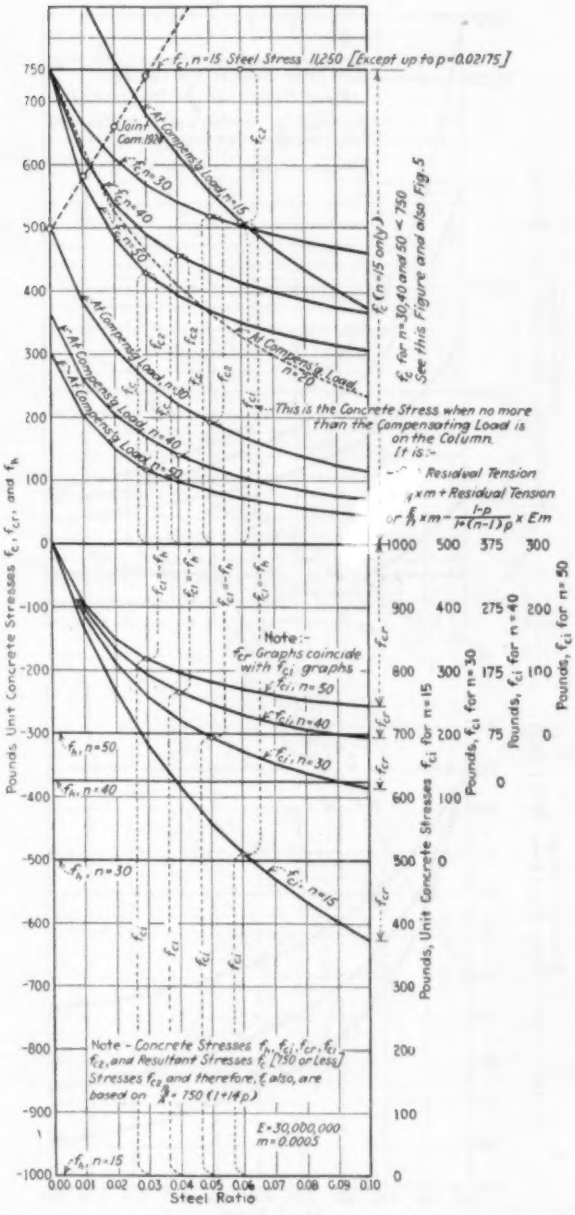

f_c, n=15 Steel Stress 11250 [Except up to p=0.02175]
Joint Com. Ref.
At Compensating Load n=15
f_c n=30
f_c n=40
f_c n=50
At Compensating Load n=20
At Compensating Load, n=30
At Compensating Load, n=40
At Compensating Load, n=50
f_{c2}
f_{ci}
This is the Concrete Stress when no more than the Compensating Load is on the Column. It is:-
Residual Tension
$f_c \times m$ + Residual Tension
or $\frac{f_c}{1+(n-1)p} \times E_m$
f_c (n=15 only)
f_c for n=30, 40 and 50 < 750 See this Figure and also Fig. 5
Note:- f_{cr} Graphs coincide with f_{ci} graphs
f_{ci}, n=50
f_{ci}, n=40
f_{ci}, n=30
f_{ci}, n=15
f_h, n=50
f_h, n=40
f_h, n=30
f_h, n=15
f_{cr}
Note - Concrete Stresses f_h, f_{ci}, f_{cr}, f_{c1}, f_{c2}, and Resultant Stresses f_c [750 or Less] Stresses f_{c2} and therefore, f_c also, are based on $\frac{f_c}{2} = 750\,(1+4p)$
E=30,000,000
m=0.0005
Pounds, Unit Concrete Stresses f_c, f_{cr}, and f_h
Pounds, f_{ci} for n=15
Pounds, f_{ci} for n=30
Pounds, f_{ci} for n=40
Pounds, f_{ci} for n=50
Steel Ratio

DIAGRAM 4.—Concrete Stresses f_h, f_{ci}, f_{cr}, f_{c1}, f_{c2} and Resultant Stresses f_c for $n = 15$, 30, 40 and 50.

[$f_c = 750$ for $n = 15$]

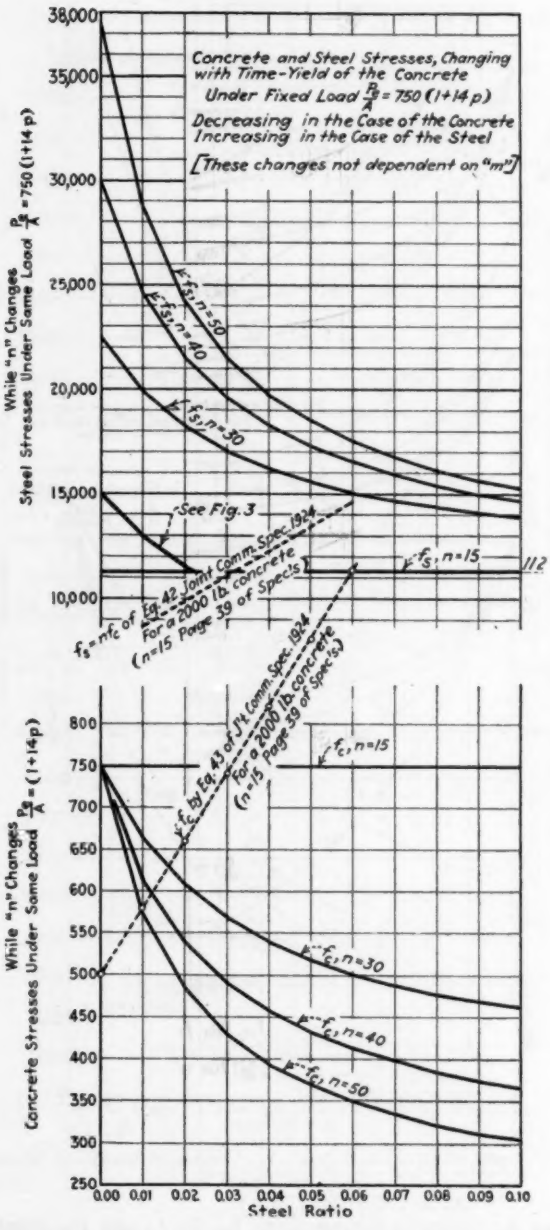


DIAGRAM 5.—Concrete and Steel Stresses, under External Loads $\frac{P_s}{A} = 750$ $(1 + 14\ p)$ corresponding to values $n = 15$, 30, 40 and 50.

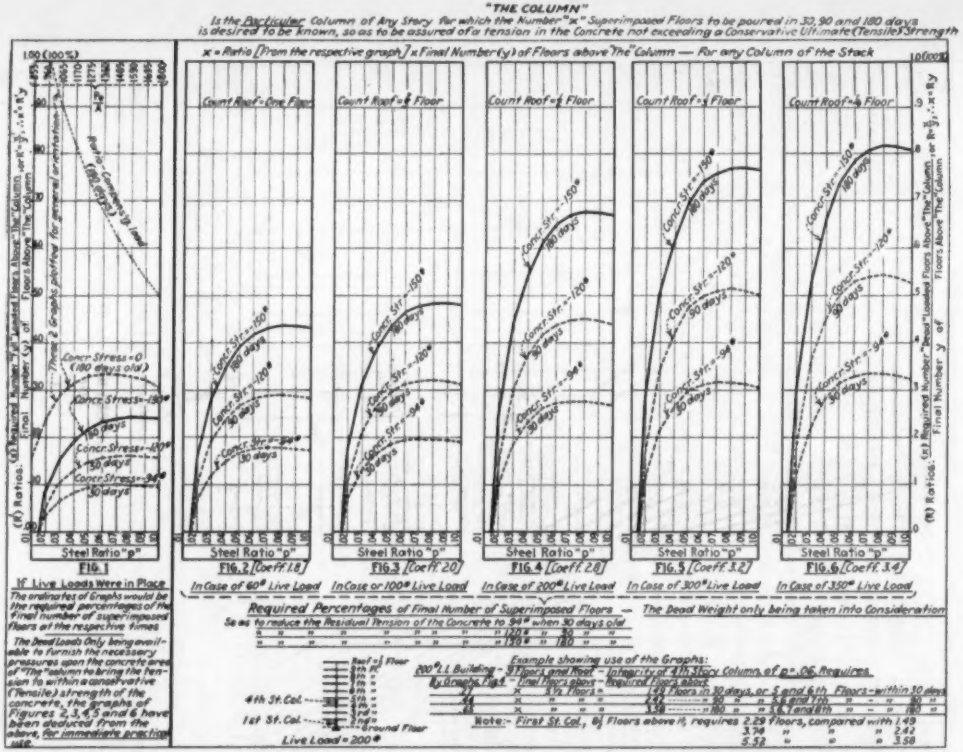

"THE COLUMN"
Is the Particular Column of Any Story for which the Number "x" Superimposed Floors to be poured in 30, 90 and 180 days is desired to be known, so as to be assured of a tension in the Concrete not exceeding a Conservative Ultimate (Tensile) Strength
x = Ratio [From the respective graph] x Final Number (y) of Floors above "the" Column — For any Column of the Stack
Count Roof = One Floor
Count Roof = ½ Floor
Count Roof = ¼ Floor
Count Roof = ⅛ Floor
Count Roof = 1/16 Floor
Concr. Str. = 150# 180 days
Concr. Str. = 120# 90 days
Concr. Str. = 94# 30 days
Steel Ratio "p"
Fig. 1
Fig. 2 [Coeff. 1.8]
Fig. 3 [Coeff. 2.0]
Fig. 4 [Coeff. 2.8]
Fig. 5 [Coeff. 3.2]
Fig. 6 [Coeff. 3.4]
In Case of 60# Live Load
In Case or 100# Live Load
In Case of 200# Live Load
In Case of 300# Live Load
In Case of 350# Live Load
If Live Loads Were in Place
The ordinates of Graphs would be the required percentages of the final number of superimposed floors at the respective times.
The Dead Loads Only being available to furnish the necessary pressures upon the concrete area of "the" column to bring the tension to within a conservative (Tensile) strength of the concrete, the graphs of Figures 2, 3, 4, 5 and 6 have been deduced from the above, for immediate practice use.
Required Percentages of Final Number of Superimposed Floors — The Dead Weight only being taken into Consideration
So as to reduce the Residual Tension of the Concrete to 94# when 30 days old
Example showing use of the Graphs:
200# L.L. Building — 9 Floors and Roof — Integrity of 4th Story Column, of p = .06, Requires
By Graphs, Fig. 4 — Final Floors above — Required Floors above
.03 x 5 to Floors = 1.50 Floors in 30 days, or 5 and 6th Floors within 30 days
.04 x " " = 2.47 " " 90 " " 5 & 6 and 7th " " 90 "
.06 x " " = 3.58 " " 180 " " 5, 6, 7 and 8th " " 180 "
Note:- First St. Col., 6 Floors above it, requires 2.29 Floors, compared with 1.49
3.74 " " " 2.47
5.52 " " " 3.58
Live Load = 200#

DIAGRAM 6.—(Appendix II) Required percentages of final number of superimposed floors, to reduce concrete tension to conservative ultimate stress, with floors rated at their design weights and also with floors rated at their dead weights only, for 30, 90 and 180 days old concrete.

$n = 15, f_c = 750$]

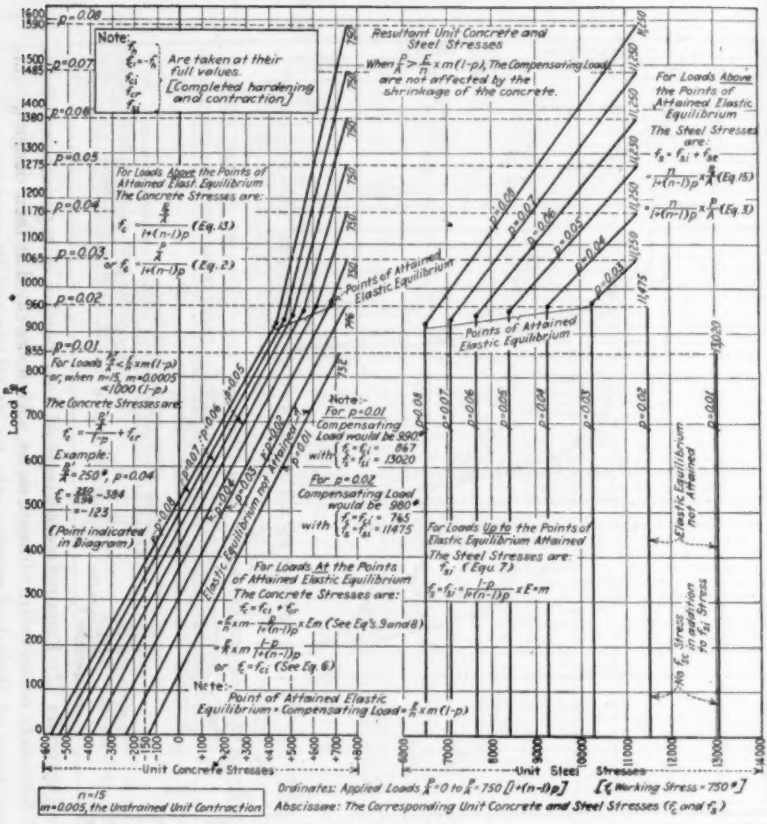

Resultant Unit Concrete and Steel Stresses
When $\frac{P}{A} > \frac{E}{n} \times m(1-p)$, The Compensating Loads are not affected by the shrinkage of the concrete.
Note: f_c, f_{ci}, f_{cr}, f_{si} Are taken at their full values. [Completed hardening and contraction]
For Loads Above the Points of Attained Elast. Equilibrium The Concrete Stresses are: $f_c = \frac{\frac{P}{A}}{1+(n-1)p}$ (Eq. 13) or $f_c = \frac{P}{A[1+(n-1)p]}$ (Eq. 2)
For Loads $\frac{P}{A} < \frac{E}{n} \times m(1-p)$ or, when n=15, m=0.0005 <1/1000 (1-p) The Concrete Stresses are: $f_c = \frac{\frac{P'}{A}}{1-p} + f_{cr}$
Example: $\frac{P'}{A}$=250#, p=0.04 $f_c = \frac{250}{.96} - 384 = -123$ (Point indicated in Diagram)
For Loads At the Points of Attained Elastic Equilibrium The Concrete Stresses are: $f_c = f_{ci} + f_{cr} = \frac{P}{A} \times m - \frac{P}{A[1+(n-1)p]} \times Em$ (See Eq's. 9 and 8) $= \frac{P}{A} \times m \frac{1-p}{1+(n-1)p}$ or $f_c = f_{ci}$ (See Eq. 6)
Note:- Point of Attained Elastic Equilibrium = Compensating Load = $\frac{P}{A} \times m(1-p)$
Elastic Equilibrium not Attained
Points of Attained Elastic Equilibrium
Note:- For p=0.01 Compensating Load would be 990# with $f_c' = f_{ci} = 867$, $f_s' = f_{si} = 13020$
For p=0.02 Compensating Load would be 980# with $f_c' = f_{ci} = 765$, $f_s' = f_{si} = 11475$
For Loads Up to the Points of Elastic Equilibrium Attained The Steel Stresses are: f_{si} (Equ. 7) $f_s' = f_{si}' = \frac{1-p}{p(n-1)p} \times E \times m$
For Loads Above the Points of Attained Elastic Equilibrium The Steel Stresses are: $f_s = f_{si}' + f_{sc}$ $= \frac{n}{1+(n-1)p} \times \frac{P}{A}$ (Eq. 15) $= \frac{n}{(1+(n-1)p} \times \frac{P}{A}$ (Eq. 3)
No f_{sc} Stress in addition to f_{si}' Stress
Elastic Equilibrium not Attained
Load
Unit Concrete Stresses
Unit Steel Stresses
n=15 m=0.005, the Unstrained Unit Contraction
Ordinates: Applied Loads $\frac{P}{A}$=0 to $\frac{P}{A}$=750 [1+(n-1)p]
Abscissae: The Corresponding Unit Concrete and Steel Stresses (f_c and f_s')
[f_c Working Stress = 750#]

DIAGRAM 7.—The unit resultant concrete and steel stresses corresponding to external loads, as these loads increase from $\frac{P'_e}{A} = 0$ to $\frac{P'_e}{A} = 750\ (1 + 14\ p) = \frac{P_e}{A}$

$[n = 15]$

APPENDIX I

Distribution of Stresses Along Column Shaft

The stresses f_h (eq. 4), f_{ct} (eq. 6), f_{si} (eq. 7), f_{cr} (eq. 8), all due to the contractive effort of the concrete, are the *maximum* stresses in the middle of the column shaft.

When the concrete is not restrained *at all*, or only partly so, all particles move toward the middle, the lengths of their movements are proportional to their distances from the middle. Since the individual movements of all particles are made in the same length of time, their respective velocities are likewise proportional to their distances from the middle. The *kinetic* energies of the particles when they move, or their *potential* energies when they are held, are proportional to the *squares* of their velocities, and, therefore, proportional to the *squares* of their distances from the middle. The force which resists the movements of the particles toward the middle, is the adhesion of the concrete to the steel (the bond). This force is exerted against the potential energy of the particles, and, therefore, whatever it is, we know it must be proportional to the *square* of the distance from the middle.

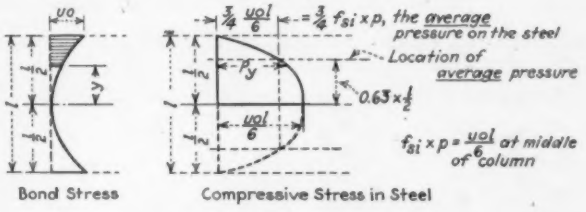

uo
y
Bond Stress
¾ uol/6 = ¾ fsi × p, the average pressure on the steel
Location of average pressure
0.63 × l/2
uol/6
fsi × p = uol/6 at middle of column
Compressive Stress in Steel

Fig. A

If u is the bond stress, per sq. in. of the contact area, at the ends of the column, o the perimeter of the steel in the unit size ($1\ \square''$) reinf. column (being the sum of the perimeters of all the steel verticals in the column of core area A divided by said A), then uo is the bond stress per lin. in.

The bond stress at the distance y from the middle is to the bond stress at the ends as $y^2 : \left(\frac{l}{2}\right)^2$. The bond stresses are the abscissae of a parabola with its vertex at the mid-length of the shaft. Its equation is $y^2 = \frac{\left(\frac{l}{2}\right)^2}{uo} x$. The adhesion from y to the end of the column is the cross-hatched area between y and $\frac{l}{2}$.

The *total* bond stress from either end to the middle of the column is $\frac{uol}{6}$.

This latter amount is, therefore, the *whole* pressure, *at the middle of the column*, upon the steel of area p, and also the *whole tension* on the concrete

of area $1-p$, (the latter only as long as there is no external load on the column, $\frac{P'_e}{A} = 0$)

The pressure upon the steel p (or the pull on the concrete $1-p$, when *no* external load) at the distance y is

$$P_y = \frac{uol}{6}\left[1 - \left(\frac{2y}{l}\right)^2\right]$$

as shown in Fig. A. The maximum, at the middle of the column, is $\frac{uol}{6}$. [The average pressure is $0.75 \times \frac{uol}{6}$ at a distance of $0.63 \times \frac{l}{2}$ from the middle of the column.]

We have at the middle of the column

$$\frac{uol}{6} = -f_{cr}(1-p) = f_{si} \times p = \frac{p(1-p)}{1+(n-1)p} Em, \qquad \text{solved for } u.$$

$$u = 6\frac{p(1-p)}{1+(n-1)p} \times Em \times \frac{1}{6} \times \frac{1}{l} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

This equation gives us the bond stress at the ends of the column due to the contractive effort of the concrete, and this bond stress *does not* change during the application of an external load $\frac{P'_e}{A} \leqq \frac{E}{n} m(1-p)$, because during this period the steel stress f_{si} and the total shear in one half the length of the column are both *constant*.

Demonstration of *constant* total shear, in the half length of column, while $\frac{P'_e}{A}$ increases from 0 to $= \frac{E}{n} m(1-p)$: (see Fig. B).

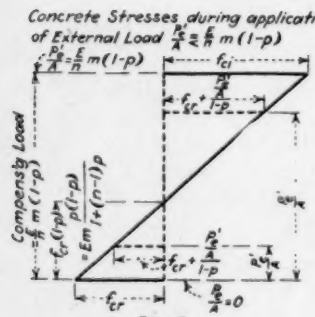


Fig. 3

$E = 30{,}000{,}000 \quad n = 15 \quad m = 0.0005$

STEEL RATIO	STRESSES			LOADS		
				Concrete str. = 0	Concrete str. = f_{ci}	Design Loads
	f_{si}	f_{cr}	f_{ci}	$-f_{cr}(1-p)$	$\frac{E}{n} m(1-p)$	
.01	15030	-132	867	131	990	855
.02	11500	-234	765	229	980	960
.03	10260	-316	680	307	970	1065
.04	9230	-384	615	369	960	1170
.05	8380	-440	557	418	950	1275
.06	7650	-488	510	459	940	1380
.07	7050	-529	470	492	930	1485
.08	6500	-565	433	520	920	1590

$$f_{si} = \frac{1-p}{1+(n-1)p} \times Em$$

$$f_{cr} = \frac{-p}{1+(n-1)p} \times Em$$

$$f_{ci} = \frac{1-p}{1+(n-1)p} \times \frac{E}{n} m$$

FIG. B

During the application of the external load $\frac{P'_e}{A} \leqq \frac{E}{n} m(1-p)$ the steel stress is constant, $f_{si} = \frac{1-p}{1+(n-1)p} \times Em$. The steel stress f_{si} is due to

the adhesion of the concrete to the steel while the concrete tends to contract. We have

$$-f_{cr}(1-p) = f_{si} \times p \quad \text{when } \frac{P'_e}{A} = O$$

$f_{si} \times p$ is *constant* while the external load increases from $\frac{P'_e}{A} = O$ to $\frac{P'_e}{A} = \frac{E}{n} m(1-p)$.

Therefore, the shear on the concrete side of the bonding surface must be *constant* also, and, equal to $-f_{cr}(1-p)$.

(1) For $\frac{P'_e}{A} = -f_{cr}(1-p)$ the concrete stress is O, but in its place the external load $-f_{cr}(1-p)$ tends to slide the concrete along the steel surface toward the center of the column, just as the pull in the concrete does when $\frac{P'_e}{A} = O$.

(2) When $\frac{P'_e}{A} < -f_{cr}(1-p)$ the force tending to slide the concrete along the steel surface toward the center of the column is made up of a pull in the concrete (toward the center) and the load $\frac{P'_e}{A} < -f_{cr}(1-p)$.

The pull in the concrete is $-\left[f_{cr} + \frac{\frac{P'_e}{A}}{1-p}\right] \times (1-p) = -f_{cr}(1-p) - \frac{P'_e}{A}$.

The external load tending to slide the concrete along the steel surface toward the center is $\frac{P'_e}{A}$.

The sum of the two forces is $-f_{cr}(1-p)$.

(3) When $\frac{P'_e}{A} > -f_{cr}(1-p)$ the force tending to slide the concrete along the steel surface toward the center is the external load $\frac{P'_e}{A}$ minus the compressive stress on the area $1-p$ $\left[f_{cr} + \frac{\frac{P'_e}{A}}{1-p}\right] \times (1-p)$

The difference of the two forces is $-f_{cr}(1-p)$.

When we know the length of the column and the steel perimeter o per unit size ($1\square''$) of the reinforced column we can calculate the maximum unit bond stress u (at the end of the column), by equation 17.

TABLE C.

	$E = 30{,}000{,}000 - n = 15 - m = 0.0005 - f_c = 750$	Steel Ratio							
		0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08
1	$12 - 1''\phi$ – steel area $\square''$	9.425	9.425	9.425	9.425	9.425	9.425	9.425	9.425
2	Core area A $\square''$	942.5	471.25	314.16	235.6	188.5	157.1	134.64	117.81
3	Circumference of Spiral, inches	109	76.7	63	54.5	48.6	44.5	41	38.4
4	Bars center to center, inches	9.1	6.4	5.25	4.55	4.05	3.71	3.42	3.2
5	Perimeter, $12 - 1''\phi$, inches	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7
6	Perimeter per unit size ($1\ \square''$) col. 0	0.04	0.08	0.12	0.16	0.20	0.24	0.28	0.32
7	$u = \frac{6 f_{cr}(1-p)}{o} \times \frac{1}{l} = 6\frac{p(1-p)}{1+(n-1)p} \times Em \times \frac{1}{o} \times \frac{1}{l}$	$\frac{19545}{l}$	$\frac{17250}{l}$	$\frac{15390}{l}$	$\frac{13830}{l}$	$\frac{12570}{l}$	$\frac{11475}{l}$	$\frac{10575}{l}$	$\frac{9750}{l}$
8	u_{12} for col's. 12′ (144″) long (including Comp. Loads)	136	120	107	96	87	80	73	68
9	Design Load $\frac{P_s}{A} = 750(1+14\,p)$	855	960	1065	1170	1275	1380	1485	1590
10	Compensating Loads $\frac{E}{n} m\,(1-p) = 1000(1-p)$	990	980	970	960	950	940	930	920
11	B = Design Load – Compensating Load	(−135)	(−20)	95	210	325	440	555	670
12	$f_{st} = \frac{1-p}{1+(n-1)p} \times Em$ [for loads up to $= \frac{E}{n} m\,(1-p)$]	13030	11500	10250	9230	8380	7670	7050	6520
13	f_{se} for Load B $= \frac{n}{1+(n-1)p} \times$ B	....	....	1003	2030	2870	3580	4200	4730
14	u'_{12} for Load B $= \frac{f_{se}}{f_{st}} \times U_{12}$	....	....	10.4	20.7	29.6	37.6	44	49.5
15	Total u_{12} for full Design load [lines 8 and 14 added]	136	120	117	117	117	117	117	117

11250, when $p > .0217$

Design Load > Compensating Load

Table C has been prepared, selecting eight (8) columns, each with $12-1''\phi$ verticals, varying the core size A of the column so that the $12-1''\phi$ bars represent from 1% to 8% of the column areas A. Line 7 gives the end bond stresses for the columns (eq. 17 above) as functions of the column length l. Line 8, gives the end bond stresses v_{12} for columns 12 ft. long. Line 14 gives the additive bond stresses for the excess of design loads over compensating loads for the same 12 ft. long columns. Line 15, finally, gives the *total end bond stresses* corresponding to the *full design loads*. When $p > 0.0217$, these total end bond stresses are *constant* for all values of p. These bond stresses are inversely proportional to the length of the column, *greater* when length < 12 ft., smaller when length > 12 ft. For short columns the bond stresses become rather large, the need of an effective bond surface becomes obvious.

NOTE.—It must be understood that lines 8, 14 and 15 of Table C apply to a "lone" column, i.e., one that stands on some foundation and *terminates* 12 feet above the said foundation, and *all* its external load applied to its upper surface. When other columns are superimposed, making a *stack* of columns, the length of the *stack* is the l of eq. 17 and determines the maximum bond stress (due to the contractive effort of the concrete) at the upper and lower end of the *stack*. The corresponding bond stresses at any other place are the abscissae of the parabola of Fig. "A," l being the height of the *stack*. The values of line 8 *divided* by the number k of columns in the stack are then the maximum bond stresses (due to the contractive effort of the concrete) at the upper and lower end *of* the *stack*. The *total* bond stress at the upper end of the stack would then be

$$\frac{u_{12}}{k} + u'_{12}$$

EXAMPLE.—8 columns in stack ($k=8$), all columns 12′ long, $p = 0.03$ for all columns, *total* bond stress at the *upper* end $\frac{107}{8} + 10.4 = 23.8\#$, as against $117\#$ of the "lone" 12′ long column. The *total* bond stress at the *lower* end may be assumed to be $\frac{107}{8} = 13\text{–}\frac{3}{4}\#$. If all the columns were 9′ instead of 12′, the *total* bond stress would be $23.8 \times \frac{12}{9} = 31.7\#$ at the *upper* end of the stack, and $13\text{–}\frac{3}{4} \times \frac{12}{9} = 18\#$ at the *lower* end (Note: the bond stresses u'_{12} of line 14, due to load B (see line 11) have been figured assuming that the steel share $f_{se} \times p$ of this load is transmitted from the concrete to the steel *in the upper half length* of the column, in the same manner as $f_{si} \times p$ due to the contractive effort of the concrete, so that the total steel stress in the middle of the columns is the *maximum* $f_{si} \times p + f_{se} \times p = (f_{si} + f_{se}) \times p = f_s \times p$.

As stated above, the average unit pressure on the steel (or the average pull on the concrete, when $\frac{P'_s}{A} = O$) is 75% of the maximum. This average unit pressure occurs at about $\frac{1}{6}$ of the column length from each end, it is $0.75 \times f_{si}$, and this *average* pressure causes the deformation of the *column*, thus as an *example*, the 4% steel column, under external load $\frac{P'_s}{A} = O$ up to $\frac{P'_s}{A} = \frac{E}{n} m\,(1-p)$ may be expected to show a deformation from its unhardened to hardened state

$$\epsilon = \frac{0.75 \times f_{si}}{E} = \frac{0.75 \times 9230}{30{,}000{,}000} = 0.0002308$$

and under its full design load (maximum $f_s = 11{,}250$)

$$\epsilon = \frac{0.75 \times 11250}{30{,}000{,}000} = 0.0002813$$

being for a 12 ft. long column 0.0332″ and 0.0406″, respectively.

[Greater deformations in the course of time, if such, are due to a flow of the concrete. Smaller deformations would indicate a smaller n.]

The steel and concrete stresses *at the middle of the column*, the effects of the contractive effort of the concrete, may be determined by the rate of deformation *at that place*.

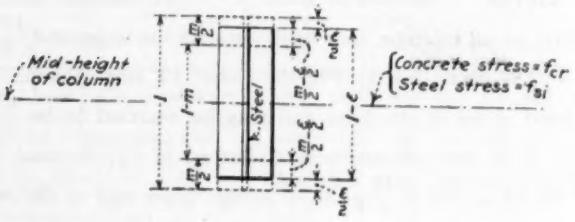


FIG. C

Let Fig. C represent a relatively short length of the unit size (1 □″) reinf. column, half above and half below the middle of the column.

E = Modulus of elasticity, steel.

$\frac{E}{n}$ = Modulus of elasticity, concrete.

1 = length of *reinforced concrete* prism before contraction occurs, or if, by some *external* means, the concrete were *fully* restrained from contracting during and after hardening.

m = the *unrestrained* unit contraction of a *plain concrete* prism at completed hardening.

$1-m$ = length of the *plain concrete* prism after contraction, or length of the concrete in the *reinforced concrete* prism if we could assume the absence of *all* bond. (Both concrete stress and steel stress = O after contraction, the concrete would slip along the steel.)

ϵ = the unit contraction of the *reinforced concrete* prism at completed hardening (bond between concrete and steel, and stress in both concrete and steel).

$1-\epsilon$ = length of the *reinforced concrete* prism at completed hardening.

The unit *tensile stress* of the concrete when *fully* restrained from contracting during hardening, or when stretched from its contracted length $1-m$ to length 1, the unit extension being $\frac{m}{1-m}$, is

$$f_h = -\frac{E}{n} \times \frac{m}{1-m} \quad \ldots\ldots\ldots\ldots\ldots\ldots (a)$$

The unit *tensile stress* of the concrete, when the prism $1-m$ is stretched to length $1-\epsilon$, the unit extension now being $\frac{m-\epsilon}{1-m}$, is

$$f_{cr} = -\frac{E}{n} \times \frac{m-\epsilon}{1-m} \quad \ldots\ldots\ldots\ldots\ldots\ldots (b)$$

The unit *compressive stress* of the steel when hardening of the concrete and contraction of the *reinforced concrete prism* is completed, is

$$f_{si} = \epsilon \times E \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (c)$$

The concrete and steel stresses at any cross-section of the reinforced concrete column balance each other, therefore, at the middle of the column

$$f_{cr}(1-p) + f_{si} \times p = O$$

or

$$-\frac{E}{n} \times \frac{m-\epsilon}{1-m}(1-p) + \epsilon E \times p = O \ldots (d)$$

Eq. *d* solved for ϵ gives:

$$\epsilon = m \frac{1-p}{1+[n(1-m)-1]p} \quad \ldots\ldots\ldots\ldots (e)$$

Since m is a relatively very small value compared with unity (for $m = 0.0005$, $1-m = 0.9995$, or 99.95% of unity), we may substitute *1* for $1-m$.

Eq. *a* becomes $f_h = -\frac{E}{n} m \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (a')$

Eq. *e* becomes $\epsilon = m \frac{1-p}{1+(n-1)p} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (e')$

Eq. *b* becomes $f_{cr} = -\frac{E}{n}(m-\epsilon)$

and substituting ϵ from eq. *e'* into the latter equation, we have

$$f_{cr} = -Em \frac{p}{1+(n-1)p} \quad \ldots\ldots\ldots\ldots (b')$$

Eq. *c* becomes, by substituting ϵ from eq. *e'*

$$f_{si} = \frac{1-p}{1+(n-1)p} \times Em \ldots\ldots\ldots\ldots\ldots (c')$$

Eq. a' is identical with eq. 4 of the original development, and eqs. b' and c' are identical with eqs. 8 and 7, respectively.

The unit deformation at the middle of the column (the maximum) given by eq. e' above is, of course, $=\frac{f_{ci}}{E} = m \times \frac{1-p}{1+(n-1)p}$ (see eq. 7), identical with eq. e' above, since f_{ci} is the *only* stress in the steel. It has been shown how the shear (in the half length of column) during the variation of the external load from $\frac{P'_e}{A} = O$ to $\frac{P'_e}{A} = \frac{E}{n} m(1-p)$ is *constant* in agreement with a *constant* steel stress f_{ci} during the same range of the external load. It remains to be shown that the deformation of the concrete, as a function of its *resultant* stress, is also *constant* and equal to that of the steel, during the same range of the external loads.

The deformation of the concrete in the reinforced column is effected by the force causing the natural (unrestrained) unit shrinkage m of the concrete, in conjunction with the residual tensile stress f_{cr} in the concrete. The unit contractive force causing an actual positive (shortening) deformation m, in the absence of any external load and of any steel, is $\frac{E}{n} m$.

In the case of the reinforced column, with an external load $\frac{P'_e}{A}$ on it, this full contractive force (a tension in the concrete) is *reduced* by the unit pressure $\frac{P'_e}{A}$ of the external load to

$$\frac{E}{n} m - \frac{\frac{P'_e}{A}}{1-p} \qquad (18)$$

it varies with the load $\frac{P'_e}{A}$.

[This *reduced* unit contractive force (18)
becomes $= \frac{E}{n} m$, when $\frac{P'_e}{A} = O$
and becomes $= O$, when $\frac{P'_e}{A} = \frac{E}{n} m(1-p)$]

The stress in the concrete also varies with $\frac{P'_e}{A}$, it is $f_{cr} + \frac{\frac{P'_e}{A}}{1-p}$ (see Fig. "B") ..... (19)

[This stress (19) becomes $= f_{cr}$, when $\frac{P'_e}{A} = O$ (see Fig. "B")
and becomes $= f_{cr} + \frac{E}{n} m = f_{ci}$, when $\frac{P'_e}{A} = \frac{E}{n} m(1-p)$ (see Fig. "B")]

The *resultant* of the reduced unit contractive force (18) and the internal stress (19) is

$$\frac{E}{n}m + f_{cr} = \frac{E}{n}m - \frac{p}{1+(n-1)p} \times Em$$

reducing to

$$= \frac{E}{n}m\frac{1-p}{1+(n-1)p} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

This is the f_{ci} of Equation 6.

This *resultant* stress is seen to be *independent* of the external load while said load varies from $\frac{P'_e}{A} = O$ to $\frac{P'_e}{A} = \frac{E}{n}m\ (1-p)$ and is, therefore, ***constant*** for the said range of the external load.

The deformation due to the *resultant* unit contractive stress (20) is

$$\frac{\frac{E}{n}m\frac{1-p}{1+(n-1)p}}{\frac{E}{n}} = m\frac{1-p}{1+(n-1)p} \quad \ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Eq. 21 is identical with eq. e' above, and, it should be borne in mind, is the maximum deformation at the middle of the column.

Example: for $p = 0.04$ we have $f_{cr} = -384$, $f_{si} = 9230$, $f_{ci} = 616$

$$\epsilon = m\frac{1-p}{1+(n-1)p} = 0.0005 \times \frac{0.96}{1.56} = 0.000308 \text{ (by eq. } e' \text{ or 21)}$$

the *resultant* concrete stress is (term 20)

$$= \frac{E}{n}m\frac{1-p}{1+(n-1)p} = \frac{E}{n} \times 0.000308 = 2{,}000{,}000 \times 0.000308 = 616$$

being the sum of $\frac{E}{n}m = 1000$ and $f_{cr} = -384$ $(= 616)$

Checking by the steel stress $f_{si} = 9230$, we have

$$\epsilon = \frac{9230}{30{,}000{,}000} = 0.000308$$

It may be asked how, while the concrete has a *tensile* stress $f_{cr} = -384$, its deformation comes to be 0.000308, a *decrease* in length. Here is the answer: If the concrete in the 4% steel column, with no external load on it, were unrestrained in its spontaneous shrinkage, its deformation would be $m = 0.0005$, and at completed deformation its stress $= O$, but the contraction of the concrete is stopped when its deformation $= 0.000308$, the unattained part of the unrestrained contraction is $0.0005 - 0.000308 = 0.000192$, and *it* causes the tension in the concrete of $-0.000192 \times 2{,}000{,}000 = -384$. This is merely another demonstration of the *reduced* contractive force of the concrete in the reinforced column to be the algebraic sum of its *full* contractive force $\frac{E}{n}m = 1000$ and $f_{cr} = -384$ $(=616)$, or $f_{ci} = f_{cr} - f_h$ (see also eq. preceding eq. 8), and $\frac{616}{2{,}000{,}000} = 0.000308$. Attention is here called to the

fact that, with no external load on the column, the tensile stress of the concrete (—384) would surely cause rupture of the concrete. When the hardening of the concrete is practically completed, we should have not less than $\frac{P'_c}{A} = (384 - 150) \times (1-p) = 234 \times 0.96 = 225\#$ or $\frac{225}{1170} = 19\%$ of its design load 1170# on the column at that time. (Refer to Diagram 7.) Appendix II gives a full treatment of this requisite for the assurance against excessive tensile stress in the concrete.

NOTE.—It might be stated, for the sake of completeness, altho merely a repetition of the development, that when the external load $\frac{P'_c}{A} \geqq \frac{E}{n} m(1-p)$, or in other words $\geqq$ the compensating load, the maximum deformation (at the middle of the column) is given by $\frac{f'_c}{\frac{E}{n}} = \frac{f'_s}{E}$, where $f'_c \geqq f_{ci}$ and $\leqq f_c$, f_c the permissible stress of the concrete, and $f'_s \geqq f_{si}$ and $\leqq f_s$, $f_s = n f_c$.

APPENDIX II

A Study of the Steel Ratio "p" of the Column

(Table A and Diagram 6 apply specially to this Chapter)

When we look for the reasons which must or should govern the relative amount of the vertical steel in the cross-section of the column let it be understood:

(a) A is the circular cross-section within a steel hooping as determined by Equation (1) assuming $f_c = 750$ lb. and $n = 15$,

(b) 1. The anti-contractive tensile stress f_h of the concrete (Equation 4).

2. The internal load $\frac{P_i}{A}$ (Equation 5).

3. The compressive stress f_{ci} of the concrete (Equation 6) and the compressive stress f_{si} of the steel (Equation 7), both due to the said internal load,

4. The resultant of f_h and f_{ci} = the residual tension f_{cr} of the concrete (Equation 8),

are based on $E = 30{,}000{,}000$, $m = 0.0005$ and $n = 15$, for the case of the 180-day old concrete (disregarding for the present any later plastic flow, represented by an increase of n).

Considering the case of $p = 0.01$ (the lower limit of the Joint Committee Specifications) we find $f_{si} = 13{,}030$ lb., on Diagram 3. The steel is stressed—*by the internal load alone*—to more than 15×750 by eq. 3, while the concrete is stressed — by the *external* load bearing on the concrete of the *residual tension* f_{cr} — to $f_{cr} + \frac{\frac{P_e}{A}}{1-p} = -132 + \frac{855}{0.99} = -132 + 864 = 732$, this is less than $f_c = \frac{855}{1.14} = 750$ by eq. 2, elastic equilibrium is *not* attained. [Only when $p \geqq 0.02175$ have we elastic equilibrium under the "design" load $\frac{P_e}{A} = 750\ (1+14 \times \geqq 0.02175)$]. If we wish to attain elastic equilibrium in a column of $p = 0.01$, the "design" load would have to be $\frac{P_e}{A} = 868\ (1 + 0.14) = 990$, in a column of $p = 0.02$, $\frac{P_e}{A} = 766\ (1+0.28) = 980$, etc. The minimum steel ratio $p = 0.01$ probably generally occurs where a column, to comply with the prescribed slenderness ratio, is larger in cross-section than required by Equation (1), so that the column load $\frac{P_e}{A}$ will be represented by perhaps $450\ (1+14 \times 0.01)$ instead of $750(1+14 \times 0.01)$. But it must be realized that a reduction of the external load to less than the load permissible by Equation (1) does not affect the steel stress *at all*, because said stress, being due to the *internal load only* (the latter a function of E, n, m, and p), *is independent of any change in the external load.* Therefore, a *reduction* of the final external load *increases* the elastic disparity between concrete and steel.

However, it is safe to say that the bond between steel and concrete is more than ample to resist the shear.

Now, considering the case of $p = 0.06$ (the upper limit of the Joint Committee Specifications), we *have* elastic equilibrium between the concrete and steel, but (for fully hardened concrete) f_{cr}, the residual tension = 488 lb. This *tensile* stress in concrete exceeds its ultimate tensile strength. Assuming say 150 lb. as a *conservative ultimate tensile strength* of fully hardened concrete we see that the required fraction of the external load must be such as will put a pressure of 488–150 = 338 lb. on the sq. in. of concrete to reduce its tensile stress to 150 lb. This required unit pressure by the external load is less than $f_{ci} = \frac{E}{n} m = 1000\#$ (eq. 9) [the unit pressure by the *first* part of the external load which part bears on the concrete area $1-p$ *only*]. The fraction $\frac{P'_e}{A}$ of the whole external load $\frac{P_e}{A}$ required to be in place on the unit size (1 □″) reinforced concrete column, causing this unit pressure of 338# on the concrete, is $\frac{P'_e}{A} = (1-p) \times 338 = 0.94 \times 338 = 318\#$, and since 1380# is the "design" load, $\frac{P'_e}{A} = \frac{318}{1380} \times \frac{P_e}{A} = 0.23 \times \frac{P_e}{A}$. Therefore, 23% of the "design" load $\frac{P_e}{A}$ must be in place when the fully hardened concrete is not to have a greater tension than 150# per sq. in. Expressing the external load in terms of the number of floors resting upon the column in question we can say 23% of the contemplated *final* number of floors, if these floors were *carrying the live loads* for which allowance has been made in the "design" load $\frac{P_e}{A}$, must be in place. But, as we have *only* the dead load part of these floors in the unfinished bare concrete structure we have to multiply the above percentage with $\frac{D+L}{D}$, where D represents the weight of the concrete floor structure (including the weight of one story of the column stack) and L the live load, on the floor area supported by the column. Diagram 6, Fig. 1 shows the required ratios of the *final* number of superimposed floors—*with their live loads included*—and Figs. 2, 3, 4, 5, 6 show the equivalent ratios of the *final* number of superimposed floors—*counting only their dead weight*—to furnish the required amount of external loads, for buildings designed for live loads of 60 lb., 100 lb., 200 lb., 300 lb., and 350 lb., respectively. The coefficients $\frac{D+L}{D}$ have been assumed as 1.8, 2.0, 2.8, 3.2 and 3.4 for the above live loads respectively, see foot of Table A.

With the compressive strength of the 180-day old concrete called 100%, the compressive strength after 30 days has been assumed as $\frac{5}{8}$ or 62½%, and after 90 days as $\frac{8}{10}$ or 80%. The tensile strength has been assumed to vary with the compressive strength. The accompanying Table A should be referred to as to the method and process of finding, successively, the values of

n' and n'', the moduli of elasticity $\frac{E}{n'}$ and $\frac{E}{n''}$, the anti-contractive tensions f'_h and f''_h, the pressures upon the concrete f'_{ci} and f''_{ci} by the internal loads, the residual tensions f'_{cr} and f''_{cr} of the 30-day and 90-day old concrete, respectively, all by the proper modification of the corresponding values for the 180-day old concrete. The last compartment of the Table shows the respective pressures on the sq. in. of the concrete to be furnished by the external load so as to bring the concrete *tensile* stresses to the permissible conservative ultimate tensile strengths $\frac{5}{8} \times 150 = 94$ lb., $\frac{8}{10} \times 150 = 120$ lb., and 150 lb. for the 30-day, 90-day and 180-day old concrete, respectively. From the required pressures per sq. in. on the concrete the loads per unit (1 sq. in.) column are derived, and these loads are further stated as the percentages of the respective "design" loads, for which latter see Diagram 1 or 2. Figs. 2, 3, 4, 5 and 6 of Diagram 6 give the information needed *for direct and practical use and application.*

We can record the following observations from these graphs:

1. The *heavier* the live load for which we have to design the *greater* the required ratios of the number of "dead load" floors which must be in place above "the" column in 30, 90 or 180 days.

2. The *increments* of all ratios from a $p = 0.06$ to a $p = 0.08$ column (0.08 is approximately the peak of all the graphs) are *less than 4% of* the ratios of the $p = 0.06$ column for all 5 cases plotted, therefore, there is no valid reason why the designer should be limited to $p = 0.06$, if he finds a complete saturation by the plastic concrete of all the space around the steel skeleton of a $p = 0.08$ column entirely practicable.

3. Taking Fig. 2, the type of Office, Apartment and Hotel buildings, a column of $p = 0.08$ requires 18, 29 and 46% of the *final* number of floors above "the" column to be in place in 30, 90 and 180 days, respectively. Suppose the building is 12 stories and a basement, or 13 floors (counting the roof as equivalent to one floor, in this case), the basement column would call for $0.18 \times 13 = 2.3$ floors in 30 days, $0.29 \times 13 = 3.77$ floors in 90 days, and $0.46 \times 13 = 5.98$ floors in 180 days, a quite reasonable expectation, especially so when we know that one of these required floors is poured simultaneously with the basement column reducing the number of *further floors required* in 30, 90 and 180 days to 1.3, 2.77 and 4.98, respectively.

4. Considering the 4th story column of this same building, also of $p = 0.08$, it would call for $0.18 \times 9 = 1.62$ in 30 days, $0.29 \times 9 = 2.6$ floors in 90 days, and $0.46 \times 9 = 4.15$ floors in 180 days, or practically the pouring of 0.62, 1.6 and 3.15 floors in the subsequent 30, 90 and 180-day periods. So, as the building gets higher, the progress may be slowed up some, as it probably is, the *fastest* progress is needed for the *lowest* tier of columns in the interest of safe-guarding the concrete against an excessive *tensile* stress, because the number "y" of floors constituting the "design" load on "the" column is *greatest* for the *lowest* tier and the required number "x" of floors above "the" column varies with the *final* number "y" of floors.

5. Taking Fig. C, representing a manufacturing building or a warehouse of say 8 stories and a basement, or $8\frac{1}{2}$ final floors above the basement column

(see Diagram 8 for accounting for roof), the basement column of $p = 0.08$ would call for $0.277 \times 8\frac{1}{2} = 2.36$, $0.45 \times 8\frac{1}{2} = 3.82$, $0.678 \times 8\frac{1}{2} = 5.75$ floors in 30, 90 and 180 days, respectively, or rather 1.36, 2.82 and 4.75 floors will actually have to be poured in the next 30, 90 and 180 days.

6. Taking Fig. 6, representing a heavy warehouse of say 5 stories (no basement), the 1st story column of $p = 0.08$ would call for $0.335 \times 4\frac{1}{4} = 1.42$, $0.545 \times 4\frac{1}{4} = 2.32$, and $0.818 \times 4\frac{1}{4} = 3.47$ floors in 30, 90 and 180 days, respectively, or rather 0.42, 1.32 and 2.47 floors will actually have to be poured in the next 30, 90 and 180 days.

All the above requirements are well within common present-day time-performances.

NOTE.—The number of floors obtained from the graphs in the manner shown above should be considered as the minimum. When more than *Ry* (see Diagram 6) floors are poured in 30, 90 and 180 days, the concrete tension is *less* than 94 lb., 120 lb., and 150 lb., respectively, and the integrity of the concrete is assured to a correspondingly greater degree.

When we give attention to an increasing deformation of the column due to plastic flow of the concrete (time-yield), we find the steel stresses gradually *increasing* with *increasing* deformation. It is true, the stress f_{si} (due to the internal load) *decreases* with an *increasing* plastic flow of the concrete, the *total steel stress* f_s, however, *increases* with an *increasing* plastic flow of the concrete (see Diagrams 3 and 5).

For $p = 0.01$, f_s increases from 13,030 for $n = 15$
to 20,000 " $n = 30$ ($53\frac{1}{2}\%$ increase)
to 24,600 " $n = 40$ ($88\frac{1}{2}\%$ ")
to 29,000 " $n = 50$ (122% ")

The deformations increase at the same rate as the steel stresses, for $n = 50$ the deformation is more than twice the *initial* deformation. If we should find the deformation to increase to as much as 3 times the *initial*, a steel stress of 39,000 lb. would result. To guard against such a contingency a steel ratio *higher* than 0.01 would appear advisable. Taking $p = 0.025$, for instance, we would have an increase of the steel stress f_s

from 11,250 for $n = 15$
to 17,600 " $n = 30$ ($56\frac{1}{4}\%$ increase)
to 20,600 " $n = 40$ (33% ")
to 22,900 " $n = 50$ (104% ")

The deformation might *possibly* be 3 times the *initial*, resulting in a steel stress of 33,750 lb. as against 39,000 lb. for $p = 0.01$ (n approximately = 195, and f_c reduced to approximately 173 lb.).

Now considering a *high steel ratio*, viz. $p = 0.08$,

f_s increases from 11,250 lb. for $n = 15$
to 14,400 lb. " $n = 30$ (28% increase)
to 15,400 lb. " $n = 40$ (37% ")
to 16,100 lb. " $n = 50$ (43% ")

and since the total time-deformation could *not* exceed $1\frac{3}{4}$ times the initial, the *highest possible* steel stress if concrete should "flow" so much as to become worthless as a carrying component would be $\frac{1590}{0.08} = 19{,}875$ lb. ($77\frac{1}{2}$% increase).

Giving due consideration to an eventual plastic time-flow of the concrete the final conclusions would be:

1. The criterion for a *low* steel ratio is the *steel stress*, for a high steel ratio the *concrete* (tensile) *stress*, which has to be counteracted sufficiently by the application of external load.

2. The use of a minimum steel ratio of $p = 0.02$ or 0.025 should be recommended, because the steel stress due to the *internal load alone*, as also its *increase* due to a "time-flow" of the concrete is considerably less than it is for $p = 0.01$ (see Diagrams 3, 5, 6 and the figures given above).

3. The use of a steel ratio $p > 0.06$ up to as much as the compact filling of the column form will allow is virtually just as safe a design as the use of $p = 0.06$ (by the latest Joint Committee Specifications), both with respect to the *assurance against an excessive tensile stress* of the concrete, and also with respect to an *increase of the steel stress* for the later plastic flow of the concrete; in fact, the *higher* the steel ratio the *less* the increase of the steel stress due to this latter cause. The upper limit of the steel ratio may be omitted altogether, the designer should be free to use his discretion as to the "practical" limit. The *higher* the steel ratio the *greater* the assurance of a *permanently more dependable* column.

DESCRIPTIVE LIST OF TABLES

TABLE A.—(Appendix II) Concrete stresses f_{ci}, f_{cr}, etc., for concrete 30, 90 and 180 days old, being data for graphs of Diagram 6.

TABLE B.—The concrete and steel stresses in three (3) reinforced columns of $p = 0.01$, $p = 0.04$ and $p = 0.06$

corresponding to external loads increasing by $\frac{1}{10}\frac{P_e}{A}$

from $\frac{P'_e}{A} = O$ to $\frac{P'_e}{A} = 750(1 + 14\,p) = \frac{P_e}{A}$

being—in *A column:*

Stresses computed by general equations 2 and 3 hypothetical for

$$\frac{P'_e}{A} < \frac{E}{n}\,m(1-p)$$

but real for $\frac{P'_e}{A} \geqq \frac{E}{n}\,m(1-p)$

in B column:

"Wrong" Resultant Concrete and Steel Stresses, an illogical compounding of the compressive stresses by the external loads (eqs. 2 and 3 presupposing a non-shrinking concrete) with the "residual" tensile stresses f_{cr} in the case of the concrete, and with the compressive stress f_{si}, in the case of the steel.

in C panel:

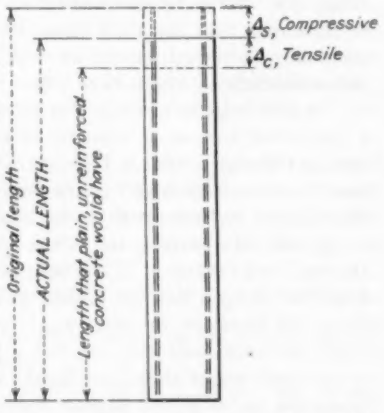
Concrete { the "residual" tensile stresses compounded with the unit pressures by the external loads

Steel {
- a, the apportioned stresses f_{si}, as long as $\frac{P'_e}{A} < \frac{E}{n} m(1-p)$
- b, the apportioned stresses by the *internal* load plus the apportioned stresses by the 2d part $\left[\frac{P_e}{A} - \frac{E}{n} m(1-p)\right]$ of the *external* loads when $\frac{P_e'}{A} \geqq \frac{E}{n} m(1-p)$

DISCUSSION.

H. M. HADLEY (*By Letter*).—Mr. Markmann's paper, although not specifically postulating continuous, unbroken bond between steel and concrete, implicitly makes this basic assumption in the development of his equations. But how the statements on which the further development of Mr. Markmann's theory depends can be reconciled with the assumption of continuous, unbroken bond, is not apparent to the writer. Mr. Hadley.

The accompanying figure represents a reinforced-concrete column, after shrinkage, but without external load. For simplicity, the entire deformation is indicated as having occurred at one end. The total steel deformation, $\triangle_s$ is shown and the corresponding total concrete deformation $\triangle_c$, is likewise shown, this latter being the difference between the actual length, and the length which plain unreinforced concrete would have.



If now this column is subjected to an external axial compressive load, either in a building, or in a compression machine, or in any other manner, a further shortening of its length must ensue, which further shortening, so long as the bond between steel and concrete is unimpaired, must of necessity be a simultaneous shortening of both steel and concrete.

But shortening of the column *increases* the total steel deformation, at the same time it is reducing the concrete deformation, which change, under Hooke's law, can only be interpreted as an increase in steel stress at the same time that the stress in the concrete is being reduced. Not until all tensile stress is removed from the concrete can it support any external compressive load and when the concrete begins to withstand compression, the total steel deformation has increased from $\triangle_s$ to $\triangle_s + \triangle_c$. Hence, the difficulty with Mr. Markmann's theory which otherwise would relieve the steel of stress and would improve the reinforced-concrete column.

M. B. LAGAARD (*By Letter*).—In these calculations a number of assumptions have been made regarding the joint behavior of concrete and steel which serve as a basis for the development which follows. This conception of this physical relation between the steel and the concrete is original and interesting, and if the assumptions as stated at the beginning of the paper are accepted the remained of the paper must be accepted Mr. Lagaard

since the calculations follow each other logically. These views, however, are not in accord with those formed by the writer from his studies of shrinkage and expansion during the past twelve years.

Mr. Markmann points out that concrete shrinks during the process of hardening and drying so that after a period of curing in air the length of an unrestrained plain concrete member is somewhat less than its original length. He also states that if the specimen is restrained, the tendency to shrink nevertheless exists and the result is the development of tension in the concrete.

In the case of a plain concrete column, it is stated there is no resistance to contraction and, therefore, there is no tension in the concrete. In other words, the column merely shortens as shrinkage progresses. It is further stated that in a reinforced-concrete column the steel, being bonded to the concrete, by its compressive strength tends to resist the contraction of the concrete. There can be no misunderstanding up to this point. The reasoning, however, is further carried on by the statement that since the steel cannot be put in compression without engaging the concrete to cooperate with it, the resistance to the contraction of the concrete is the resistance offered by the reinforced-concrete column. The reinforced-concrete column, not the steel alone, it is said, resists the contraction of the concrete. This view cannot be held, however, if considered in the light of the deformations which take place under these conditions.

It will help in making this thought clear if the condition of stress in a restrained section of concrete is assumed to take place by the concrete first shrinking to its full unrestrained value and then being pulled by tensile forces at each end to its original length. The total tensile stress in the concrete is then equal to the load necessary to produce this stretching.

If the two extremities of the specimen when fully stretched are now assumed to be fastened to the ends of a steel rod and released, the steel rod would be thrown in compression and would shorten. As shortening took place, the stress in the steel would increase and the stress in the concrete would decrease until they reached a state of equilibrium. The total stress in the steel would then just equal the total stress in the concrete and the numerical value would depend upon the amount of steel and the elastic properties of the materials.

That this is actually what takes place in reinforced concrete may be seen from the consideration of the test of a double reinforced-beam made at the University of Minnesota in 1914. This double beam consisted of two 4 x 5 x 48-in. beams reinforced with two ⅜-in. round bars. The beams were cast about ½ in. apart on their sides with the reinforcement placed in the two exterior vertical faces. They were anchored together at the two ends by means of stirrups cast in the specimens. These served somewhat as hinges. Measurements were taken across the two beams, at the center and at the two points of anchorage. The readings at the two ends showed practically no movement but the reading across the beams at the center showed a very rapid separation indicating a "bowing out" at the middle.

In other words, there was a deflection of the beams, concave on the side without reinforcement. This would seem to indicate that the steel reinforcement reduced the shrinkage on one side while the other side was free to shrink, and thus a deflection occurred as shrinkage progressed.

Tests of two beams cast at the same time, but cured differently, will also be of interest in this respect. Both beams were 4 x 5 x 48 in. in cross-section and were reinforced with a single ⅜-in. round rod. One beam was cured wet and the other dry. Measurement taken from the age of one day indicated that the wet beam had expanded slightly and the steel was in tension while the dry beam has contracted considerably throwing the steel into compression. The two beams were loaded to beyond the ultimate tensile strength of the concrete and during this procedure extensometer readings were taken on the steel bars.

The deformations in the steel of the air-cured beam were found to be several times the measured movement in the steel of the moist-cured beam, thus indicating that there must have been a difference in the original state of stress in the two steel reinforcing rods, since the quality of the concrete could not make that much of a difference. Beyond the point where the concrete had cracked, the movements were about the same in both beams. In view of these tests and the explanation of the results, it appears that the fundamental conception of the behavior of a reinforced-concrete member during the process of shrinkage, as stated in the paper under discussion, is not in accord with experimental data, and the formulas developed on this hypothesis are, therefore, likewise at variance.

The assumption that the concrete (as well as the steel) resists the action of shrinkage when it is part of a reinforced beam and does not resist shrinkage when the steel is left out, indicates a change in the properties of the concrete due to the presence of the steel rods. Again the assumption that the full shrinkage tensile stress $(\frac{E}{n}m)$ is applied to the column, which could only occur when there is no deformation in the column due to shrinkage, seems also beyond justification.

Likewise the description of the concrete being stressed, and, therefore, deformed by an external load, while the steel, which is bonded to the concrete, remains stationary or neutral, is a somewhat unique conception of the interaction of the two. It does not seem logical that these deformations could take place in one and not the other without affecting the bond between the two materials.

The conclusion that the strength of the column for carrying its external load is just the same as if no shrinkage occurred, is, therefore, based on assumptions that have not been established as facts and which contradict results of measurements on columns in actual service. The writer is, therefore, compelled to disagree with Mr. Markmann in his theories regarding the effect of shrinkage until he produces actual test results verifying his theories.

Mr. Godfrey.

EDWARD GODFREY (*By Letter*).—The reinforced-concrete column is one of the most important elements of design, if not the most important, before the engineering profession. It is beyond question that inadequacy of columns in reinforced-concrete buildings has been responsible for nearly all of the wreckage that has stained the short history of reinforced concrete. One may question the proposition that design has been responsible for the failures and blame it on materials or workmanship, but no one can question the fact that column weakness is the explanation of the large extent of wreckage once a failure has started.

It behooves engineers to look carefully into the design of reinforced concrete and not to ignore or suppress any arguments that place the blame squarely on designers. Until designers furnish plans of reinforced-concrete structures that cannot be assailed, they have no right to investigate the methods of construction; and until every objection to any system of design is met by something better than ridicule and suppression, the profession is guilty of propagating unsound methods of design.

I consider Mr. Markmann's paper an attempt to justify and perpetuate a type of design that has been assailed and has never been publicly and openly defended. I refer to the standard methods of designing reinforced-concrete columns, particularly the rodded column; for, while in Table C the author refers to a spiral, this is the only mention of anything but a shaft of concrete and upright rods as elements of a reinforced-concrete column.

This paper is purely a theoretical investigation. It does not attempt to justify results by citing confirming tests. Such procedure is fraught with the gravest danger. Based on the properties of materials as discovered by tests on small specimens a column formula that sees no further than simple compression can give nothing but misleading results.

A few decades ago column formulas for cast-iron columns were derived on the basis of the strength of cast-iron specimens in compression, and a liberal factor of safety was allowed. Tests and experience proved that the allowed load, in spite of the liberal factor of safety, was too great. Cast iron will resist, in short blocks, 100,000 lb. per sq. in. in compression. It will resist 20,000 lb. per sq. in. in tension, which is a necessary property of a column material. Cast iron is thus 50 times as strong in compression and 100 times as strong in tension. Yet the unit stress used on cast-iron columns is only about ten times as great as recommended by Mr. Markmann and by standard authors and committee reports and building codes for reinforced-concrete columns. Do we need to seek further for an adequate explanation of the fact that acres upon acres of floors supported by such columns and millions of cubic feet of buildings have collapsed in ruins, before any superimposed load was placed on the floors?

Prof. George F. Swain, in a paper read in October, 1926, before the American Institute of Steel Construction, says, "I have strong conviction that a large tensile strength is very desirable even in a matter which is to be exposed only to compression." Prof. A. N. Talbot, a number of years

ago made a similar remark, and it was made a special item of news in the *Engineering Record.* Years before that time I had published the same thing in articles and books (see my book "Concrete," 1908, p. 6).

I believe that standard literature will be searched in vain for any mention of this essential fact of column investigation. It is far from being sufficient to look merely into the compression strength of a column material in order to arrive at its value in a column. This is amply proven by the history of cast-iron columns. One failure of consequence, however, (see *Engineering News*, March 10, 1904) was sufficient to convince metal designers of the unreliability of cast-iron columns in a high building. Thirty or forty such failures have had but little effect on reinforced-concrete designers.

The reason why a column material to be efficient must be strong in tension is because the compressive strength of a round or square column depends on the shearing strength of the material, and shearing strength is always great or small according as the tensile strength is great or small. Not long ago a paper was published by the American Society of Civil Engineers which set forth the fact that concrete columns fail in shear, which is a function of the tensile strength of concrete, and not by simple compression. I was consulted before the publication of this paper and strongly recommended that it be accepted for publication because of what it would doubtless develop, if freely discussed. However, when it came to discussion of the paper the publication committee refused to let me discuss it. This is one of the many ways in which discussion of reinforced-concrete design is suppressed, another explanation of the great number of failures.

The commonly held notion regarding compression in a concrete column is that every square inch in the cross-section is of equal value. The following two quotations illustrate this:

"It may be well to repeat that the longitudinal rods must take stress. They are not the cause of failure as has been intimated. Neither does concrete fail in compression, as indicated by the author of this paper, by shearing and spalling out in chunks. Such condition comes after the maximum load has been reached." (This is by Prof. A. N. Talbot and was spoken in a discussion of the writer's paper read before the American Concrete Institute in Feb., 1915. This discussion has not yet been published.)

"A unit volume of concrete has the same ultimate strength no matter where it is placed, and it should be permitted an invariable working stress. Its position in the structure does not change its safe stress." (This is quoted from John Tucker and is in a paper by him in *Transactions* Am. Soc. C. E., Vol. LXXXVI, 1923.)

In the paper read before the Am. Soc. C. E., already referred to, almost the first statement made by the authors is this:

"Plain concrete columns * * * fail in shear rather in compression."

This is not an incidental statement; it is the keynote of the paper.

Almost the last statement made by the authors is this:

"Longitudinal reinforcement, consisting of rods or bars should not be used unless it is well tied together at frequent intervals with lateral bands.

It should preferably not be used except in conjunction with helical reinforcement." Between these two quoted statements there is theoretical demonstration of both of them; and both of them are contrary to established fact and established practice.

As a corollary to the proposition that compression specimens of concrete fail and exhibit theoretical weakness in shear, it can be proven in a very simple manner that the concrete around the periphery of a circular cross-section is weaker in resistance than the concrete located toward the center of the specimen.

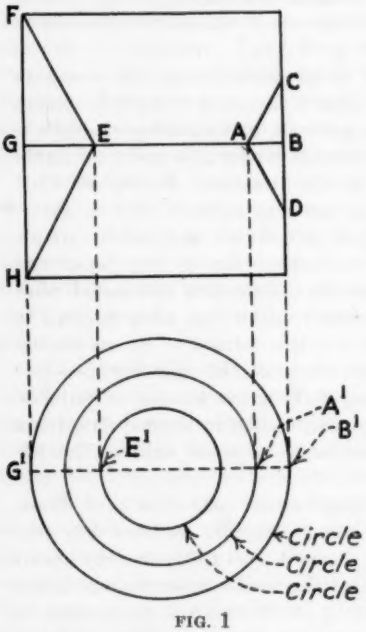


FIG. 1

In Fig. 1 the relation between the annulus bounded by circles one and two and the frustum of a cone whose element is AC is equal to the relation between the annulus bounded by circles one and three and the frustum of which EF is an element. If no other elements of strength were present, a unit compression on the first annulus would produce the same unit shear on the first frustum as the same unit compression on the second annulus would produce on the second frustum; and if such were the case, sectional area near the surface in a circular cross-section would be as effective in resisting compression in the specimen as interior area. But there is another element of strength and one of vital importance, though it is rarely given any recognition in engineering works. It is the ring tension that tends to hold the material of the specimen against the shearing or spalling action.

Assuming GE twice as long as AB the compression on the annulus represented by GE will be less than double that on the annulus represented by AB. But the area HEF in ring tension is four times as great as the area DAC. In other words, as we recede from the surface of a circular section, the resistance offered by the ring tension increases at a greater rate than the square of the distance. This is why superficial spalling or shear failures commonly occur in compression tests. This is why the interior unit volumes of compression specimens and compression members are so much more effective in resisting compression than unit volumes near the surface, contrary to Mr. Tucker's statement, above quoted, which is the general attitude taken by authorities and not the dictum of one writer. The foregoing demonstration proves why a material weak in tensile

strength is weak as a column, and conversely a material of high tensile strength as well as high compressive strength is strong in columns.

That concrete compression specimens do spall off around the periphery and break out before the specimens finally break or even receive their ultimate load is the common observation of those who have watched tests made. The writer has frequently seen this exhibited. Very frequently the fractures are such as to leave two truncated cones, demonstrating clearly that the shearing action is the cause of the failure and not crushing. Manifestly the small interior circle of concrete in compression and not hampered by shearing weakness would not long sustain the ultimate load of the column, and where the spalling due to shear is comparatively large, simultaneous failure of the small area would be expected in spite of its greater unit resistance. If the entire load of a specimen having 100 sq. in. of sectional area were suddenly thrown on the center core of 10 sq. in. by the spalling away of the 90 sq. in., naturally the over-burdened core would break with the suddenness that would leave no time for observation of any interval.

That concrete is weak in shear is a fact well attested by experiments and experience. The unit value as given in books runs all the way from 10 per cent of the compressive unit strength to 100 per cent. Experiments that demonstrate the low values are made on specimens where the concrete is not confined nor specially reinforced against tension. This is the condition to be expected in most structures. It is the condition in plain concrete and in rodded columns. Experiments that seem to show anything approaching the high value are so hedged about with precautions that preclude all possible tension in the concrete that the results mean scarcely anything to the designer.

In the following respects standard practice—"established fact"—is at fault:

(a) In failure to recognize and emphasize the fact that compression members such as concrete columns are weak in resistance to compression because of the inherent weakness of the material in shear.

(b) In failure to recognize the fact that this weakness causes failure of columns in detail and accounts for many failures at ridiculously low unit stresses.

(c) Where (a) has been recognized, the consequence exhibited in (b) has been ignored.

(d) Unit compression values of columns are invariably based on the compressive strength of the concrete in short blocks and totally regardless of the shearing strength.

(e) In classing a rodded column as a reinforced-concrete column.

The peripheral weakness of a concrete compression member is the true criterion of its value for supporting loads. Failure to recognize this and to take it into account and to reinforce against it has resulted in a large amount of poor designing and a large number of failures of structures.

Another phase of the peripheral weakness of plain or rodded columns, if such columns were worthy of consideration, is that theoretically at least, the round sections of smaller diameter are weaker per unit area than larger columns, for the reason that the peripheral area in the smaller ones is relatively greater than in the larger ones. The writer believes that this has never been discussed, inasmuch as the universal idea concerning compression members in concrete is that every square inch of the cross-section is of the same value. This is probably the explanation of the fact for example, observed by McKibben and Merrill, (*Proceedings* American Concrete Institute, 1916, p. 209):

"The average compressive strengths of 8 x 16-in. cylinders and 5-ft. columns, 14 in. in diameter, are very nearly equal."

Columns do not have the same compressive resistance as short cylinders. In the case just cited it just happened that the reduction due to column length balanced the increase due to greater diameter of cross-section.

Unquestionably the rodded concrete column (a concrete column having embedded in it straight, slender rods and no spiral or close-spaced hooping) is a universal standard. No standard book condemns it, and all standard specifications hold it out as a proper and accepted type of column.

Recently some of the reports on the Japanese and Santa Barbara earthquakes have spoken vaguely about buildings of antiquated types of design having failed and have very significantly recommended that all columns *in earthquake zones* be spirally reinforced. No hint is given as to what is antiquated type in this very young branch of designing. But if the rodded column is antiquated, it has very suddenly become so.

Mr. Markmann states many times in his paper that the vertical steel rods in a column receive no added compression besides that which is put upon them by the shrinkage of the concrete, until the unit load on the column is such as to render zero the initial tension in the concrete due to the same shrinkage and the constraint of the steel. In other words a large endwise load may be placed on a concrete column in which vertical rods are embedded, and the steel will receive none of that load. If this is true, the column does not shorten under this load application. For if the column shortens, the embedded steel must shorten, and if the steel shortens, it must receive stress. There is no material nor combination of materials known that will not, in a column, shorten under an endwise load. Absolute rigidity is as far from engineering as a fourth dimension.

If Mr. Markmann means to convey the theoretical proposition that at the middle of the length of a column there is a spot where a balance of effects exists that neutralizes the added stress on the steel, this is a reason for the steel being apportioned more than its share of the load in some other zone than this neutral one to make up the shortening of the rods that must occur in the act of loading the column. Thus one of Mr. Markmann's contentions, and one which he so strongly and so frequently emphasizes, is rendered null and void; and the rodded column is shown to be a

combination of a weak and brittle shaft of concrete in which are embedded rods already highly stressed by the shrinkage of the concrete and still more highly stressed by the application of load on the column, the rods being held by the uncertain and precarious means of concrete that may itself be on the verge of rupture from shrinkage tension.

Mr. Markmann uses 750 lb. per sq. in. on the concrete. It is not stated that the column is reinforced with hooping or a spiral. If the column under consideration is a rodded column, and not spirally reinforced, this would be a dangerous load and not a safe load.

Mr. Markmann proposes an astounding thing in design, namely, the use of 6 per cent and more of vertical steel in columns, which will, he admits, result in shrinkage tension in the concrete that would rupture it; but a condition of using this design is that the column be loaded down in the progress of the construction to prevent this rupture. I do not think such a proposition was ever put before engineers before. What would be the result if the progress of the work were unavoidably stopped? He recommends that any upper limit of steel rod area be omitted from specifications, and he states:

"The higher the steel ratio the greater the assurance of a permanently more dependable column." And to avoid rupture in that "dependable column" the superimposed structure must mount up very rapidly. The greater the percentage of steel in a column the greater will be the burden on the precariously weak concrete to hold this steel in line.

Near the other end of the steel scale it has been shown by measurements that concrete shrinkage is sufficient to stress the embedded steel to the elastic limit, and contrary to Mr. Markmann's assertion of absolute rigidity in the column steel (while the concrete shortens) this stress is added to by the column load.

Actual tests show very discordant results as to the alleged reinforcement contributed by longitudinal steel, which is what would be expected from the erratic behavior of concrete in the matter of shrinkage. Designers are becoming suspicious of reinforced-concrete columns, particularly the rodded column, which is a moribund type of design.

Mr. Cross.

HARDY CROSS (*By Letter*).—This paper illustrates the strange fallacies into which one is led when algebraic juggling supplants logical thought. The writer of the paper has apparently begun with an appreciation of the fact that, if the bond between concrete and steel is preserved in a concrete column, the total deformation of the steel is equal to that of the concrete. At some point he has, apparently unconsciously, modified this to mean that the deformation of the two materials *due to stress* is the same. There is no basis whatever for the latter statement and consequently no basis for the formulas and curves deduced by the author.

The idea that the steel stress is increased and the concrete stress decreased as a consequence of shrinkage is clearly stated by Considere. Once pointed out, it seems obvious unless the bond of the green concrete is inadequate to maintain the stresses during the period of active shrinkage

or unless plastic deformations in the concrete make all arguments based on elastic moduli invalid. The paper, however, contributes nothing to the discussion of these questions of physical fact. The physical elements in the problem have, however, been clearly stated by Mr. MacMillan.

Mr. Richart. F. E. RICHART.—In view of the fact that Mr. Markmann has made such an elaborate analysis and has done so much work on this subject, I regret that I am forced to disagree flatly with the basis of his whole analysis, that is, the basic assumption that is made on the third page of the paper. I think that we can agree with the general proposition that shrinkage of the concrete produces tension in the concrete of the column and compression in the steel. That agrees with the theory advanced by Professor McMillan and with actual tests. As to the subsequent action produced by external load, the crux of the matter is expressed in the first paragraph of page 128 of his paper. We are quite familiar with the principle of superimposing stresses due to different effects; we superimpose bending moments, deflections, slopes, and many similar quantities to find the combined effect of different force systems. The problem treated therein is to my mind simply a matter of superimposing upon the stresses produced by shrinkage the stresses produced by compression in both the steel and the concrete. Now the statement is made in the paper that, assuming that the column already contains shrinkage stresses (tension in the concrete and compression in the steel) when external load is put on, the concrete will take all of the load and the steel will remain neutral. That disagrees with the fundamental conception of all existing analysis of the action of concrete columns. If the concrete is to take stress and the steel to remain neutral, that means that the deformation of the concrete proceeds while the steel stands still; in other words, there is slipping between the concrete and the steel. Apparently such an assumption does not proceed from a study of the elastic deformation of the two materials, acting together as we must assume in any column action. The validity of the whole paper rests upon this one assumption, which does not seem to be based upon a study of the deformations in the column or upon any test results, but is simply a bald statement of what may be assumed to occur on the ground that "as the water seeks its level, so the concrete seeks to put itself into elastic equilibrium with the steel." That does not seem to be the proper basis for analysis. I think that this assumption and the assumption of distribution of bond stresses are the two points upon which we would disagree mainly. In Appendix I the distribution of bond stress is based upon a study of the relative velocities and the kinetic energy of concrete particles which are assumed to be moving during shrinkage. Now while some use is made in the analysis of the action of elastic structures of the kinetic energy or work of deformation of the internal stresses, as in the well-known principles of Least Work and Virtual Work, I can see no evidence that these principles were applied to this case. It seems to me that the distribution of bond stresses due to shrinkage must be determined experimentally, either directly or indirectly. The extent to which slipping takes place between concrete and steel and

the effect of varying lengths of member must be known before a definite idea of the variation of bond stresses may be obtained. There is need for more experimental knowledge and fewer assumptions as to the behavior of reinforced-concrete members.

PROF. A. N. TALBOT.—I had not expected to discuss this paper, but I will say a word or two on this whole matter. This is an example, I think, of difficulties that come to writers on columns by starting out with an assumption not borne out by experimental data. Surely it cannot be true, if one would stop to think, that there be strains without stress. The load put on the column, referred to in the analysis on p. 128, shortens the column. Under the assumptions made elsewhere, the steel will shorten as well as the concrete. Surely that shortening will affect the stress in the steel as well as the stress in the concrete. That being true, the assumption upon which the analysis is based is not tenable. It follows that the analysis has nothing to stand on. That is my opinion after looking it over. I wish we did have more information on the action of reinforced-concrete columns, on the bond stresses developed and the relation between the stress put into the steel and the stress put into the concrete by shrinkage. Surely at the end there must be some slipping during the time of setting and perhaps further slipping during the time of hardening. How far that slipping extends from the end of a bar, we do not know. Information upon this action may have been obtained by someone. The condition must be different beyond the point of slip, along the bar, regardless of the amount of bond that is developed at the point of slip. Further, if the concrete in the column is restrained from free shrinkage during setting and hardening, we do not know to what extent its strength is decreased by that restraint, if any. We know that the stiffness of the concrete is decreased, its modulus of elasticity has been decreased by such restraint. At least that is one of the conclusions that we seem to be able to draw from many tests that have been made in years back. Attention may well be called to another aspect of such discussions. I feel that we are making a mistake and are studying the question rather academically when consideration is given only to what is happening at working stresses. Instead we should consider what will happen at the critical load, that load upon which we base our factor of security. Now for the hooped column or spirally-reinforced column, the conditions attending the deformations and stresses in the column change markedly at loads beyond the loads for which the unspiraled concrete would fail in compression. The strain at the point of failure of unspiraled concrete amounts to about 0.0013, or so, corresponding to 35,000 or 40,000 lb. per sq. in. stress in the steel, if it is still within its elastic limit. Beyond that point, for the spiraled column conditions change and variations due to shrinkage have a much smaller effect, if they have any effect. Writers of papers of an analytical character generally base their discussions on nominal assumptions, the conventional assumptions of the designing room that are properly used in the designing room because they are the rules, but these nominal or conventional assumptions do not apply to the actual

Prof. Talbot.

physical behavior of the column, they do not apply to the analysis as it should be made. For this the actual phenomena involved should be studied and considered.

Mr. Markmann

MR. MARKMANN.—Professor Richart has taken exception to my assumption of the theory of distribution. According to his theory, he assumes that the shear between the concrete and the steel was distributed between the end of the middle and the straight-line basis. That would mean then that the shear at the middle of the column would make an abrupt change from one direction to the other, while this parabolic distribution is the natural distribution of the shear which in the middle of the column changes from one direction to another gradually. I would like to know whether he gave any attention to Table C of that example I worked out for the bond stress in columns. The results on the basis of parabolic distribution of shear give 117 lb. bond stress per sq. in. at the end of the column. If the column is 12 ft. long the longer column will give a greater dimension. By the parabolic distribution of shear, the bond stress at the end is greater than the bond stress Mr. Richart has by a straight-line distribution; so in that case the parabolic distribution or the shear at the end of the column is greater than under the other assumption.

Prof. Talbot.

PROF. TALBOT.—May I correct a misapprehension on the part of Mr. Markmann with respect to Bulletin 126? Mr. Richart is not the author of that paper; the author is Mr. Matsumota, of Formosa, Japan, who, in a study of quite another character, the matter of the effect produced by adding water, drying and wetting reinforced concrete, made certain assumptions as to the distribution of the bond stress along the reinforcing bar. He said in that paper that it was not definitely known what the real distribution is, but he assumed one as being perhaps the highest and as bringing the worst conditions. In the analysis he was working on, another assumption of distribution of bond would have made little difference in results.

ESTABLISHING AND OPERATING A CONCRETE BLOCK PLANT.

BY W. H. WARFORD.*

My experience in establishing and operating a concrete products plant has convinced me that there are three steps of vital importance, namely:

1. Study of local markets.
2. Preliminary investigations.
3. Economic production.

I feel that all of these should be given a great deal of consideration while the plant is being planned and that economic production and the study of local markets cannot be overlooked after the plant has opened its doors. What I have to say here is essentially a short history of my experience in building and running a plant for the manufacture of concrete block. The presentation of the problems encountered and their subsequent solutions is not given as a guide for other men engaged in the concrete products business, but rather to start a discussion among the other manufacturers present. Such a discussion, I am sure, will bring forth some interesting and valuable information.

Study of Local Markets.—For a number of years the conviction had been growing in my mind that a modern concrete block plant could be profitably maintained at Aurora, Illinois. Just about a year ago this conviction forced me to investigate the possibilities in detail, beginning with a study of the local markets. The investigation revealed that a variety of masonry materials—such as clay brick, selling at $12 per thousand and clay tile at $185 per thousand, offered competition which was expected to be of the keenest kind. In addition it was disclosed that several manufacturers were producing concrete block and selling them at approximately $170 per thousand, on the average. A survey of the building field, including a study of the jobs already under way and those that were in prospect, showed that there was ample room for another products manufacturer. Too, the combined production of the established concrete products plants seemed entirely inadequate to supply the demand, especially if the markets were properly developed and products could be made that would profitably compete with other materials.

Preliminary Investigations.—These conditions led to a further study of the situation. In particular, the probable cost of production, in the event a plant was established, was considered. Rough estimates, based

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upon published data and corrected for local costs of raw materials and labor, showed promise of successful operation. In view of the fact that standard size 8 x 8 x 16-in. concrete block are generally used throughout the state of Illinois, a decision to manufacture this type of unit was quickly reached. Since the selection of the 5 x 8 x 12-in. light-weight concrete tile or any special type of block as the main product of the plant would require an extensive and expensive campaign for introduction, such units were left off our list of products for the time being.

Literature was obtained from all leading manufacturers of concrete block machinery. This literature was followed up by salesmen, each of whom was given full opportunity to explain the various and sundry merits of his company's proposition. Numerous operating block plants were then visited and the operation of various machines was observed. Several makes of machinery were found to produce satisfactory block of high quality, give excellent production at low unit cost and stand up well in heavy service. Therefore, the chances of obtaining quick and efficient service on the machines after installation was the basis for our final selection of manufacturing equipment.

While investigating machinery, the arrangement of operating plants was carefully noted, for undoubtedly a well-arranged plant materially reduces labor cost. It is important that the total distance from the location of raw materials in storage to the finished block in the stockpile be kept as short as possible. Both floor space and labor are conserved by a compact arrangement of machinery. Storage facilities for cement should be provided to keep the cement in good condition regardless of weather and also to permit buying in carload lots. Plant accessories such as aggregate bins, heating plant and office should be located so that if enlargement of plant is found necessary, because of demands for increased production, changes can be made without disturbing the general scheme. Sufficient floor space should be provided to take care of any orders for special work and to prevent congestion of racks and pallets. Movement of materials should be handled by gravity methods wherever possible, thus overhead bins for handling aggregates are advantageous. Likewise, a concrete floor from the machines to the curing rooms and through the curing rooms to the yards should have a slight slope to lend assistance in the movement of trucks or cars loaded with finished block. Many of these features of plant layout or design were noticed in other successful plants and then incorporated in the layout and design of our present plant.

After having had considerable experience in making layouts and building material bins for other modern plants, we designed for our own use what we think is a very practical and economical layout. A concrete silo storage bin, 64 ft. high and having four separate bins or compartments, is used, the total material capacity of our storage being 360 tons of sand, gravel and limestone. We employ a track hopper, getting all our material in hopper-bottom cars, then elevating it by chain and bucket elevator and drawing it out into dump cars by gravity. From there it is carried by

machinery directly to the operators. With this arrangement one man can easily feed three mixers. In place of cars operated over a fixed track, we make use of lift trucks in moving finished products.

Particular attention was given to plant location because of its relation to the cost of delivery of raw materials as well as finished products. A railroad site was necessary because both cement and aggregates had to be handled in carload lots. As we looked for such a site we also had in mind that it would be possible to ship a limited amount of block by rail to nearby towns, although we had planned to use motor trucks in delivering a majority of the products. The site finally selected for the plant is at North Aurora, three miles from the center of Aurora, which is to the south, and from three to seven miles from three other cities located to the north. The combined population we serve is nearly 75,000. We found after the plant began operating that we could easily deliver by truck, the many concrete roads of the vicinity making such deliveries possible. In some instances, products are delivered by truck to points as far away as twelve or thirteen miles from the plant. At the time the site was selected, we anticipated the growth of Aurora to come our way and we have not been disappointed in that respect.

After the preliminary investigations were completed, it was found that the cost of producing concrete block could be more closely estimated than we at first surmised. We found that if rigid control of all manufacturing processes was maintained, we could expect reasonable profits from our undertaking.

The plant was erected in the dead of winter, the work progressing regardless of extremely cold weather. The plant was completed and ready for operation in the spring, actual production beginning on May 1 of last year. This sidelight of the establishment of the plant, I think, is more or less interesting.

Economic Production.—Low overhead costs, efficient labor, accurate cost information, maintenance of standards of quality at or below competitive cost and application of sound scientific principles to insure this quality are, in my mind, the principal factors in the successful operation of a concrete products plant. Intensive study of each of these principles was made in order to secure economy of production in our plant.

Included in the working forces of our organization, the Warford Construction Co., were some very good employes—men who had been with us for from seven to fifteen years. We could depend upon them to follow instructions to the letter and, of course, their experience in working with concrete was worth a great deal. Hence, these men were selected to form the nucleus of the working force in the new products plant. So at the outset, we had an efficient labor organization.

A system was installed by which a daily report of the activities of each employe was made. In addition, production records of various classes of products were kept, the records showing quantity of materials used and actual production costs.

Our methods of curing products are not unique, but, nevertheless, we have found them to be very practical. After very exhaustive research we gathered together all of the available information on the experiences of others in the matter of curing. We found that approximately 1.2 sq. ft. of radiation for each 10 cu. ft. of curing room space are required in practice. Of course this cannot be applied to every type. Our curing rooms are covered with wooden roofs. In providing heat for the curing rooms we employ a system of steam pipes which, incidentally, are not used in moderate weather. When the temperature is from 40 to 75 deg. F., we find that radiation is not required. Steam for curing is provided by one ½-in. wet steam pipe in which are drilled fifteen ⅛-in. holes. A separate valve on this pipe is used to control the amount of steam passing into the curing room. In moderate weather, with live steam escaping from this pipe and the heating pipes idle, we produce a temperature ranging from about 100 to 110 deg. F. A low pressure boiler is employed to provide steam for the live-steam pipe and the heating pipes.

Moisture in the plant in cold weather is a troublemaker, we find, and I would like very much to hear from someone who has found a method of counteracting this undesirable situation.

Sand, costing 50¢ per ton; pea gravel, graded from ¼-in. to ½-in. and costing 50¢ per ton, and limestone screenings, graded from 0 to ⅜-in., costing $1.68 per ton, are the aggregates available in our local market. The prices quoted are the actual costs of the materials delivered on our siding. These aggregates are purchased from commercial plants. Their quality and cleanliness are found to be excellent. A mixture of all three aggregates was selected for initial use, pending actual experiments to finally determine the best combination of aggregates. This original mixture corresponded somewhat with those used by products manufacturers in the surrounding territory. Finally, the following mixture was selected as the most adaptable for our use:

One cubic foot of limestone screenings; 2 cu. ft. of ¼ to ½-in. pebbles; 7 cu. ft. of very coarse 0 to ¼-in. torpedo sand, one sack of portland cement.

With this mixture we produce 22 block per sack of cement. Recent tests showed that this mixture produced block that meet standard specifications as to compressive strength and absorption.

With the inexpensive but excellent sand and gravel available, we found it impossible, when using an economic mix of these materials, to take the block off the machine without slump or breakage. The addition of the fine limestone to the mix corrected the difficulty. The limestone we use is extremely fine, 60 per cent passing through a 100-mesh sieve, 25 per cent through a 40 sieve and 15 per cent through a ¼-in. sieve.

In preparing this short paper, I have endeavored to portray the problems encountered in the establishment of a concrete products plant. Too, I have hoped to stimulate a discussion of plant operation which will involve an exchange of ideas, of discoveries and of solutions of problems that

have at some time or other been more or less perplexing. Of course, there are particular difficulties which can only be overcome by each individual manufacturer, but there are other major items in which we are all interested. For example, system is everything in a concrete products plant. In discussing the methods used by various manufacturers in obtaining systematic practices, we will all benefit, I am sure. Reduction of costs without reducing the quality is another mutual interest.

EFFECT OF FINELY GROUND CEMENTS ON CONCRETE PRODUCTS MANUFACTURE.

BY GEORGE D. CHANDLER.*

Before going into the practical end of this discussion it is thought worth while to quote from Professor Abram's conclusions in Bulletin 4 of the Structural Materials Research Laboratory entitled "The Effect of Fineness of Cement."

Conclusion 4 states "Fine grinding of cement is more effective in increasing the strength of lean than of rich mixes."

Conclusion 5 states "Fine grinding of cement is more effective in increasing the strength of concrete at 7 days and 28 days to one year."

Conclusion 6 states "The rate of increase in strength with fineness is lower for very wet mixes."

Conclusion 9 states "For leaner mixes and wetter consistencies finer cements showed a somewhat greater slump than coarse cements."

Fig. 7 on p. 24 of the bulletin referred to above shows strength curve for relatively lean mix using cement of various degrees of fineness. A strength gain of between 2 per cent and 3 per cent for each additional point increase in fineness of grinding is indicated. Assuming a 10-point finer grinding we have a theoretical gain in the neighborhood of 25 per cent.

If the statement that 7-day strength is increased 25 per cent for this amount of finer grinding, should we not expect a still greater percentage increase at the end of the customary 24 hours of steam curing?

If the rate of increase in strength with fineness is lower for very wet mixes is it not reasonable to assume that the reverse is true of very dry mixes?

If a greater slump is obtained with fine cements in lean and wet mixes, should we not reasonably expect a greater ease of compacting by means of mechanically applied force for the lean and dry mixes which the concrete products plant uses?

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* Superior Products Co., Detroit, Mich.

In the foregoing discussion the attempt has been made to demonstrate that the lean, dry, mechanically compacted mixes are in the class most benefited by finer ground cements and it may be well to mention here the possible advantages to be gained by high early strengths so obtained.

The item of greatest expense that enters into the cost of products manufacture is without a doubt the cement and any means of producing a greater strength with less cement and lower costs will be generally welcomed.

The mix that may be subjected to the most shock without breakage immediately after manufacture is undoubtedly the most desirable. There is evidence that this condition is more readily achieved with finely ground cements than with coarse cements.

Most manufacturers utilize steam curing processes for the purpose of economizing in manufacturing space and equipment, to reduce storage time before shipment and to allow rougher handling without breakage when moving the cured product to stock. Artificial curing in itself is expensive and is used purely to promote economy. The more strength acceleration a given cement has inherent in it, the more value the artificial curing has, the shorter the necessary curing period and the shorter the necessary elapsed time between the raw materials at the machine and the finished product on the job.

It may be of some interest to quote actual results obtained under normal manufacturing conditions in comparing two cements. The following test results were obtained during the early part of 1926:

Block Made With	Age	Crushing Strength Per Square Inch of Gross Area
Cement A	7 days	441 lb.
" B	7 "	742 "
" A	9 "	403 "
" B	9 "	682 "

During the summer of 1926 another comparison of the two cements was made under actual plant conditions. Certain changes had been made in the processes used in manufacturing cement A. Except that this cement was ground approximately 10 points finer than had been the previous practice, no information is available that would indicate any other difference in the cement. The fineness of the two cements at the second comparison was about equal and the results were as follows:

Block Made With	Age	Crushing Strength Per Square Inch of Gross Area
Cement A	5 days	700 lb.
" B	5 "	626 "
" A	8 "	732 "
" B	8 "	888 "

The foregoing tests were not made on a laboratory basis. Undoubtedly variables were introduced which prevented our getting exact quantitative results. The percentage increase in strength was very much over 25 per cent and we are inclined to attribute this further strength gain to a greater machineability of mix. We feel that the results obtained have had a distinctly beneficial effect on our manufacturing operations, both from a quality and a cost point of view. We also feel that a much deeper investigation of the subject than we have been able to give it is well warranted by the indications so far obtained.

ARRANGEMENT AND DESIGN OF CURING ROOMS.

BY AUSTIN CRABBS.*

By way of introduction, it should be stated that this paper is merely as the writer's personal opinions on steam curing based on several years' observation and study, and is given with the idea of stimulating discussion and in the hope that some of the ideas presented herein may prove of interest to other concrete product manufacturers. Most of these ideas have been tried out at the plant of the Cement Products Co. at Davenport, Iowa, and the final results, at least in this case, were satisfactory. This paper therefore will be largely a summary of the development of steam curing at this particular plant and this fact should be kept in mind in considering any of the statements herein.

Our first experience in steam curing was obtained by building four curing rooms about 4 ft. wide and somewhat over 50 ft. long with an arched roof over each, made by plastering cement mortar over arched forms, and reinforced with mesh. The steam was introduced into a water trough and no other means was provided for regulating either temperature or humidity. The use of water in the troughs was discontinued in a very short time, as the temperature and moisture seemed to stay at a better point if the steam were discharged directly into the air. It was also discovered that the steam would have to be shut off part of the time in mild weather, and altogether in hot weather, to keep the temperature from running too high, and in summer with the sun beating on these thin slab roofs, water sprayed into the rooms would soon vaporize without the use of additional heat. Our experience with these first crude curing rooms led to the conclusion that, in steam room design, more thought should be given to the problem of keeping constant the conditions within the rooms regardless of the temperature and humidity outside. As the variation in outside conditions during the year differ widely in different parts of the

* Cement Products Co., Davenport, Iowa.

country, this problem is a separate and distinct one for any given locality. For example, if the temperature in the winter goes down to 30 deg. below zero and up to 90 deg. or 100 deg. in summer, the design of steam rooms, at least for year-round operation, is a different problem than it would be for a climate where temperatures seldom go below the freezing point. Without considerable insulation, it is reasonable to assume that considerable heat will be lost in winter in even moderately cold climates, if steam rooms are built in an exposed location, and a uniformity in temperature and humidity will be correspondingly harder to maintain. Therefore, let me ask, why are steam rooms usually constructed as separate units outside the main factory building and not inside and an integral part of the factory?

We believe that the present common practice of separate rooms is the result of precedent and not the result of a careful study and investigation of proper location. If figured out in advance, the steam rooms often could be included in the main building, thus overcoming at once one of the main problems to be dealt with, namely the problem caused by a big difference in temperature at different seasons of the year, as temperatures inside the building, of course, do not have the same wide variation as outside temperatures. This applies not only to winter conditions, but to summer conditions as well, as it is often difficult to keep sufficient humidity in rooms exposed to the hot suns of summer. At first thought, it might seem that too much space in the building would be required for inside curing rooms, but most plants probably could be arranged to make use of the space above the steam chambers.

The above statements bring up the proposition as to proper heights for steam rooms, and the writer's observations in different sections of the country lead him to the belief that many curing chambers are entirely too high, as it is not an uncommon sight to see warm, moist fog up against the ceiling of even a moderately high steam room and not nearly as much moisture around the blocks, where the moisture is most needed. With lift-trucks, more headroom is probably required, but if cars are used, the ceiling can be kept down very close to the top blocks on the cars, as a man pushing a car naturally leans forward so that there is no danger of bumping his head against the ceiling. By keeping the height down as much as possible, a floor can be constructed over the top of the curing rooms to form a valuable working or storage space without getting an extreme height of the entire building, and in addition allow more headroom for the machine floor proper to provide for various items of overhead equipment which may be required and automatically giving better light and ventilation where the men are working.

As to the other dimensions for curing rooms, the width should be such that a little space, at least on one side, is left after the room is filled, as it is usually inconvenient to work in too close quarters; and in the case of the car system, it is often found necessary to get into a loaded curing room on account of an accident to one of the cars. With cars, our expe-

rience would indicate that two tracks in each curing room allowing just room enough for a man to pass between loaded cars on adjacent tracks furnish a very practical arrangement without having the walls between rooms too far apart to be floored over in an economical manner. In establishing the length for steam curing rooms it should be kept in mind that we are dealing with a heavy material and should therefore try to arrange all parts of a concrete products plant so that the materials are moved through the least possible distance. On this assumption, we have limited the lengths of our own steam rooms to accommodate five cars per track, or approximately 42 ft. long, and have further shortened up distances by having the machine room at the center of the building with steam rooms at each side, so that the finished product moves either way from the center and travels shorter distances than it would if the curing rooms were all in one direction.

While on this question of curing room size and arrangement, I would like to suggest that valuable information might be obtained by studying out the size of plant required to give maximum economy in operation, as I believe that there is a certain point with a given type of machinery wherein it would be more economical to build an entirely new and separate unit rather than to increase the size of the original plant to such an extent that distances become too great.

In designing steam rooms for heat and moisture, the temperatures and humidity of the air surrounding the rooms need to be taken into consideration, as previously intimated, and after these conditions are fairly well understood, it then becomes necessary to decide just what method is to be used for maintaining proper conditions inside the rooms. The present practice, as you know, is to use closed heating coils, live steam introduced directly into the air, or spray or fog nozzles, or any combination of these three, and curing rooms in the future may possibly adopt all three. In fact, it would not require a very big stretch of the imagination to think that some day, in view of the fact that this industry is continually becoming more scientific, that the best curing rooms will also be equipped with recording hygrometers and remote reading thermometers so that the conditions at all times will be a matter of exact knowledge. With the conditions known at all times, additional moisture and heat could be introduced with low pressure live steam; additional heat could be supplied as required by turning steam into the closed heating coils; and after the product had started to set, the temperature, if too high, could be brought down and additional moisture supplied by the introduction of cool water through the fog sprays. Just how high a temperature should be maintained is a matter for further study, but there probably is no advantage in using steam unless the temperature in the curing rooms can be brought up to at least 100 deg. With any such temperature, the humidity must be maintained at a high enough point to keep the surrounding air from absorbing moisture from the product, for, if this occurs, the product will be poor, both in color and in strength. On the other hand, too high a degree of

humidity with respect to the temperature of the product will cause enough condensation on the product itself to produce sagging or actual collapse, unless the material is allowed to take an initial set before being subjected to such conditions.

These thoughts would seem to lead to two conclusions, first, that a heating of cold materials in the mix would be advisable to allow the product the benefit of being placed in the moist rooms as soon as made, and second that some provision should be made for increasing the humidity above the dew-point after the product has taken an initial set.

A high degree of humidity in the curing rooms, of course, causes a more or less serious problem of keeping this dampness out of the rest of the plant, but this is mostly a matter of providing proper ventilation together with some method of construction at the entrances of the curing rooms that will prevent too great a leakage of steam or vapor. We have tried various systems, such as curtains, vertical sliding doors, etc., but have obtained the best service from our present method which, briefly, is as follows:

The curing rooms each contain two tracks and in order to allow access to each track separately and to avoid uncovering the entire opening at one time, each opening is equipped with two doors hinged separately at the top and fitted with a T-bar astragal between, which also hangs from a hinge at the top, so that it is free to swing up with either door and at the same time closes the opening between doors when both doors are shut. At the tops and sides, the doors close against stops and a further protection is secured by a flat plate bolted along the edge of the door to strike so as to close against the jambs and head. These stops and plates form what the steel casement window manufacturers would refer to as two-point weathering and give a construction free from leaks. The bottom does not need to close as tightly as the top and sides, and as some trouble is caused by moisture and on exterior doors by freezing, we have found it practical to fit a 1-in. strip on edge at the bottom of the door extending it from the floor to within a fraction of an inch of the bottom of the door. In our plant, the doors were made short enough to allow the use of a piece of 1 x 4-in. board, on edge, below, and if this piece swells and sticks or freezes solid on outside doors, it will break out when the door is opened without damaging the door and these small strips, in such an event, can be replaced without serious expense and by breaking, save lots of wear and tear on the doors themselves.

In a construction as described above, the key to the whole arrangement is to have a steam room door that will not swell or warp out of shape. This problem we believe we have solved by buying commercial tin-clad doors, of the fire-door type, but made to our specifications and sizes. If such doors are kept painted to protect the metal covering the metal covering in turn protects the wood cores from the action of dampness, with its attendant ills. We have used doors of this kind for three years now,

and while they will soon need another coat of paint, they have proven highly satisfactory and we cannot see where they are not as good now as when we first got them.

APPLYING SCIENTIFIC PRINCIPLES TO CONCRETE PRODUCTS MANUFACTURING.

BY C. J. HERZOG.*

Applying scientific principles to concrete products manufacturing is a subject so large that one could talk about it all morning and then only be started. An annual production of over 600,000,000 blocks in 1926, valued in excess of $100,000,000, is evidence enough that the business is rapidly becoming a basic industry. Within the last five years there has been a great change in the concrete masonry field. The capacity of machinery has been largely increased, we have learned to turn out products of a high degree of uniformity in physical properties, and are able to meet all the conventional requirements as to strength, low water absorption, load-carrying ability, wear resistance, fireproofness, etc.

With the formation of companies having greater capital investment, and with more economical plant equipment it is necessary to consider all of the possible methods of reducing production costs of concrete products. These methods might be listed under four general headings:

(1) Plant equipment and layout
(2) Gradation of aggregates
(3) Appearance of finished product
(4) Economy of organization

Much has been written about this first item, plant equipment and layout, and I do not care to enter into a detailed discussion of it at this time, except to mention that this matter should be given most careful consideration in order that labor and handling cost be reduced to a minimum.

The second item, that of gradation of aggregates, probably presents the greatest opportunity to concrete products companies to reduce the production costs of their units. The investigation conducted by the Portland Cement Association at Pittsburgh, known as the "Pittsburgh Block Tests," demonstrated clearly the advantages of a coarser grading of aggregates. The outstanding conclusion one must reach from these tests is that the coarsest possible grading should be used consistent with appearance of the products.

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These tests also seemed to indicate, though less clearly, that the addition of a small amount of fine aggregate, such as limestone screenings, makes for a more workable mixture and reduces breakage to a minimum without materially decreasing strength. Although this matter has not received sufficient investigation to determine its possibilities, the principle has been applied with highly satisfactory results in a number of plants. In order to put this principle into practice, three plants of the General Products Corporation of Pittsburgh were equipped to use three aggregates. I might mention that the results already obtained have more than justified the expenditures, and it is hoped that by a further study of grading and a closer control of the fineness modulus still greater savings can be effected. The use of three aggregates also gives a means for closer control of the fineness modulus.

The matter of appearance of concrete masonry is of such vital importance that it should receive careful consideration.

Perhaps the chief criticism and objection raised against the use of concrete masonry units for exterior walls is the monotony of color effect. Gray is not a stimulating color; it has little or no psychological value; it makes no visual appeal. We must bear in mind that the majority of people are color starved; that is to say, consciously or unconsciously, they have a hankering after color effect. The liberal use of colors in advertising is an example of proven responsiveness to color appeal; the use of colors in interior decoration is merely another example of the same thing. We are drawn to or repelled by the presence or the absence (as the case might be) of color-compelling mediums.

While the amount of color mediums that may be exposed in the exterior walls in construction of various types may represent only a small fraction of the total volume of materials used in such construction, at the same time the psychological value of this minor percentage far outweighs its percentage of volume.

With the outstanding advantages in the way of economies in use and with the proven ability to meet structural requirements, which we all know concrete products possess, there is no reason why we cannot give the same attractive surfaces to our products as can be found in any of our competitive materials. The use of facings on concrete products is relatively old, yet the percentage of faced products actually employed is relatively small. One controlling reason may be the fact that such facings have been frankly of an imitative character; that is to say, they have attempted to reproduce granite or marble surfaces, and like all imitations, the imitation has fallen below the standard of the original.

Concrete is a material sufficient unto itself. Its qualities are so outstanding that it does not have to ape any other material. It can stand on its own legs, figuratively speaking. When we, as an industry, have sufficiently grasped the importance of this idea, it is my opinion that we will be entering upon an era of expanded use of our product which will far surpass any achievements of today.

Color has been applied to concrete in many ways, which for our purpose, we will briefly mention under four headings:

(1) Colored aggregates.

(2) Colored facings applied at time of manufacturing.

(3) Colored facings applied with spray.

(4) Colored portland cement stucco.

It is common knowledge that naturally-colored aggregates, used as a facing material, will give a wide range of attractive surfaces when properly used. There are some very beautiful examples of ornamental concrete where the color effect has been secured simply by the use of such aggregates. Nearly all of the concrete trimstone manufacturers use this method of surfacing their products.

The second method, that of colored facing applied at time of manufacturing, is probably the one most generally used by products manufacturers. Its success depends on keeping the facing material wet and the moisture content constant.

The third method, that of colored facing applied with spray, is another practical way of achieving the desired result. This method has been largely confined to very fine aggregates but, I understand, recent developments have made it applicable to the use of coarser materials.

The fourth method, that of the application of colored portland cement stucco, is the one that has been most widely used and is the most practical. Plain blocks or tile are the only units that need be stocked in the yard, a fact which greatly simplifies the manufacturing problem. The units are laid more cheaply because the same care need not be taken as in laying faced units. A much greater variety of texture effects can be obtained than by any other means. The stucco, being a plastic material, does not limit the architect to design as is the case where joints must be considered.

I might mention that the stucco should in all cases be plant-mixed to give uniformly good results. There are several reasons for this—color pigments must be mixed dry to assure a thoroughly uniform color; all materials must be weighed dry to insure the same proportioning at all times; and the grading of sand must be controlled. It is obvious that these conditions cannot successfully be handled under average field practice.

I would mention, briefly, the matter of economical organization. The combination of several concrete products companies in Pittsburgh is no doubt of interest, since it was carried out with this point of view. Possibly the chief advantage presented by such a combination in a large city is the reduction of the delivery cost. With four plants situated in different sections of the city and by a careful zoning of the territory to be supplied from each, a substantial saving has been effected in delivery costs. The number of salesmen has been reduced and other items affecting the general

overhead have been cut. One of the greatest savings will result in better credit facilities afforded by a larger company.

In conclusion, I would say that the concrete products business it just coming into its own and will play an important part in helping the building industry to meet the ever-growing demand for fireproof construction.

APPLICATION OF SCIENTIFIC METHODS IN CONCRETE BLOCK MANUFACTURE.

BY BENJAMIN WILK.*

Mr. Warford in his paper "Establishing and Operating a Concrete Products Plant" has certainly covered his subject very well. Markets are important. Just as much, if not more so, than economical production. Without a market it does not matter how economically we can manufacture. If we have no outlet we will soon go out of business. While Mr. Warford has covered practically the entire field involved in the establishment of a concrete products plant, I want to emphasize economical production or the applying of scientific methods to production.

The fact is that economical production starts with the design of the plant. If the designer has not given thorough study to the problem of handling his raw materials to eliminate labor and if he does not so place his machinery that extra steps are not necessary, then the operation of the plant after the designer is through cannot be as economical as possible. In the products business the manufacturer should apply engineering principles to the manufacture of his products, and should make a thorough study of all angles of the business.

Looking at the problem from a cost standpoint it is evident that the final cost of a block consists of the following four factors:

1. Cost of aggregate per block.
2. Cost of cement per block.
3. Cost of direct labor per block.
4. Cost of overhead per block.

Cost of Aggregate Per Block.—This is an important item in the production of concrete products. It is much more important than is generally supposed, and considerable thought should be given to the location of the plant with reference to the delivery of aggregates so as to cut the cost to a minimum. In Detroit aggregate delivered by truck costs almost twice as much per ton or per yard as the same aggregate delivered by rail. On

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* Standard Building Products Co., Detroit, Mich.

the basis of 65 to 70 8-in. block to a cubic yard of aggregate the saving through the use of rail delivery means a saving of almost 2¢ per block. In considering this phase of the problem, however, we should take into consideration that most gravel pits in this latitude close down for about five months during the winter, and storage and rehandling costs money. Besides, the cost of trackage and handling equipment must be figured.

Cost of Cement Per Block.—The cost of cement per block naturally depends on how many block can be obtained from a sack of cement. Mr. Warford shows that he can obtain 22 block per sack of cement and I venture to state that there are many in this audience who are only getting 15 or 16 8-in. block from a sack of cement. On the assumption that cement costs $2.20 per barrel, or 55¢ per sack, Mr. Warford pays 2½¢ per block for cement, while some of those present here with 15 or 16 block to the sack are paying 3½¢ per block for cement. In other words, there is a possible saving of 1¢ per block in cement content. But quality must not be sacrificed. It should be emphasized that all block manufacturers should meet the requirements of the American Concrete Institute or of the municipality or state in which they are located. In many cases, also, appearance counts heavily. Some people prefer the smooth block to the rough block. They say a rough block looks porous and they have become accustomed to seeing a smooth block. But a smooth block may indicate an excess of fine material and with an excess of fine material it is hard to get 22 block per sack of cement and still meet requirements. Therefore, it is necessary to select aggregates very carefully and to combine them intelligently.

Mr. Warford has explained how he is able to obtain a large number of block per sack of cement and still meet specifications. My experience with aggregates checks Mr. Warford. At our plant we could not get a really economical mix with the available sand and pebbles even though we used an almost theoretically perfect grading. Such a grading was shown by the Cincinnati test reported several years ago by Committee P-6 of the American Concrete Institute and since referred to in Fig. 7, p. 493 of the 1925 *Proceedings*. We found that even with an experienced man and good operators, block made exclusively of sand and pebbles in a really lean mix would not stand up. The block would fall down on being taken from the machine. By substituting one-third limestone screenings of somewhat similar grading to the sand and pebbles, except that 8 per cent of the limestone passed a 100-mesh sieve, the block stood up satisfactorily and could be readily carried away from the machine. This may not check with the generally accepted theory, that aggregates of similar gradings should give similar strengths, but nevertheless it is a fact. Evidently it is necessary in making concrete products economically to use an aggregate with from 5 per cent to 10 per cent passing the 100-mesh sieve. This statement has been made before to the Institute's conventions and my experience this past year in the actual operation of a products plant bears out this theory. It would seem that the limestone screenings finer than 100-

mesh have real cementing value. On the other hand, we tried limestone screenings entirely and found that it gives too pasty or sticky a block. The appearance was not good. A sand aggregate is desirable in combination with the limestone screenings. A mixture of two-thirds of sand and pebbles with one-third of limestone screenings even of a somewhat similar grading as the sand and pebbles will give a superior product to either sand and pebbles alone, or limestone screenings alone.

In order to have a definite comparison with Mr. Warford's experiences we deliberately made up block with our regular mixer man and block operators, using a one-sack batch of 9 cu. ft. of sand graded from 0 to ⅜ with a fineness modulus of 3.9 and also a one-sack batch using 6 cu. ft. of this same aggregate and 3 cu. ft. of limestone screenings from 0 to ⅜-in. with a fineness modulus of 3.7. The aggregates were measured in a 1-cu. ft. box. Materials were mixed dry three minutes and wet three minutes. This is about the usual practice in our plant. The concrete was mixed as wet as possible as is our custom and the block were cured 36 hours in the steam kilns. The batches were made one right after the other and placed on a single rack. The block made first and last were not used for testing. The tests show that the strengths at the end of 14 days were as follows:

The sand and pebble aggregate alone, 726 lb. per sq. in. gross area;

The sand and pebble and limestone screenings, 684 lb. per sq. in. gross area.

Under the heading of cost of cement it would be well to discuss control because the better the control the less cement required per sack to obtain a specified strength.

In our plant the overhead bins are on the third story and they cover an area of 40 ft. x 70 ft. Each bin is approximately 8 ft. x 13 ft. in cross-section giving us three bays wide and nine bays long, a total of 27 bins. The mixers are placed on the second floor close to the interior supporting columns. As there are three mixers along each side of the building they are sufficiently far apart so we can, if necessary, draw aggregates from four bins into each mixer. At present we are using aggregates from three bins into one mixer. By this method we are able to get a balanced grading. We have found that it does not take long for an operator to learn how to gage, in the mixer, the right amount of each aggregate required per batch and by means of a horizontal steel slide gate below each bin opening, he can quickly control the flow from each bin. We had previously thought of measuring by weight or by volume before dumping into the mixer, but our experience so far seems to indicate that for our work we can get satisfactory results by measuring in the mixer. The operator can tell by the amount of concrete dropping on the apron of the mixer as the blades revolve whether he is getting the right amount of raw materials in each batch. It would be well to get the experiences of a number of products manufacturers on this question.

In controlling the amount of water per batch we have found that a

15-gal. tank located just above and to one side of the mixer is a reliable and desirable indicator. A glass gage on the tank makes it possible to shut off the inflow at any measured amount. A valve controls the out-flow through a perforated pipe over the mixer. This arrangement allows the operator to work on other mixers while the measured amount of water is slowly running out of the tank into the mixer. In the tests of block just referred to was used 5 gal. of water in addition to the water in the aggregate. Previous experiments showed approximately 5 per cent moisture in the sand, giving a water-cement ratio of practically 1.35. This checks very well with the reports of Committee P-6 on the tests at Cincinnati, Chicago and Humboldt, Iowa. With this amount of water web marks showed on the face and the blocks could be handled readily from the machine. We have also found that when using such an amount of water as will give a web mark that it is not a question of trying to get a certain strength that governs the number of block per sack of cement, but rather the question of using the maximum amount of aggregate per sack of cement that will allow the taking away of the block from the machine without breakage. I understand that some plants in their experiments have obtained close to 30 block per sack of cement but the breakage was so excessive that it was not considered practical. As Mr. Warford has explained, 22 block from a sack of cement can be obtained day in and day out without much trouble if attention is paid to the selection of the aggregates and the control of water. The reports of Committee P-6 of the Institute are a good guide for determining how to obtain economical production. It appears that only a comparatively few product manufacturers have availed themselves of the information—probably because they think it is too theoretical.

Cost of Direct Labor.—The direct labor cost of manufacturing the block will not vary as much as many people suppose. Operating with a good crew and one of several good machines the production per day should not vary over a wide range. We have found it best to pay on a piecework basis. Some may question the value of piecework because they feel that quality is impaired. If the piecework is a fair basis this difficulty should not exist. We have found that the production can be increased 50 per cent by paying by piecework. With such a basis we know that our labor costs will not vary from day to day and the men know that they get paid for what they produce. Good men want piecework.

Cost of Overhead.—Under cost of overhead we take into consideration all items not already covered and we include interest on investment, depreciation, insurance, taxes, light, heat and power, sales cost and advertising. The variation under this heading may be considerable. A small plant operated to capacity may have a low unit overhead, while a big plant may have a high unit of overhead, especially when the plant is running only part time.

This leads me to the thought that each plant is a study in itself. No one design will suit all conditions, nor can we say that it is usually neces-

sary to have track hoppers, elevators, overhead bins and individual overhead mixers. Because a plant has a great deal of equipment does not always mean that it is efficiently designed or operated. Certainly a one-machine plant should not be designed after the pattern of a six-machine plant, and a plant without track siding should not be designed after the pattern of a plant with track siding.

By study of locations and probably market a products manufacturer can determine how many machines to use and with this as a base he should be able to design his plant to get raw materials to the mixer economically and the finished product from the machine into the kilns and out into the storage yard at such a cost that he can sell his product at a profit. All of this means that the manufacture of concrete products is getting away from hit-and-miss methods. Plants are being built with the idea of cutting costs to a minimum and capital is being attracted because intelligent methods are being employed. Above all experience shows that there is such a thing as economical production through proper plant design and the application of scientific methods to plant operation.

SUGGESTIONS ON THE MANUFACTURE OF CONCRETE BLOCK, BRICK AND TILE.

BY NEWTON D. BENSON.*

In presenting this paper I wish to offer some suggestions on the more important phases of the manufacture of concrete products and to propose some methods which are not in very general use. In order to present these suggestions I have assumed a plant equipped to make concrete block, brick and tile and have described the features of this plant that illustrate these points.

The plant I propose to describe is planned for a growing territory with a present moderate demand for block, brick and tile. The choice of the three different units is made in order to keep the plant more uniformly in operation and also because it has been found that one product helps sell another. For example, concrete brick is frequently used to supplement concrete block and tile and even to fill in with clay tile. In fact, there is hardly a masonry building that does not create a demand for one or more of these units.

Location.—The location of this plant should preferably be at a good gravel bank and beside a railroad—as gravel can ordinarily be produced cheaper than it can be shipped, and the railroad is almost a necessity for

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* Providence, R. I.

the receipt of cement—to obtain aggregates not found in the bank and to ship the finished product.

Arrangement.—The layout of such a plant must be determined largely by local conditions. Frequently a difference in the grade of the ground can be used to advantage in eliminating the elevation of some or all of the materials, or in storing and delivering the finished product.

In the general ground plan the layout should be in what may be called "multiple units." If an initial installment of four usual-sized machines is contemplated, an outline layout for eight, twelve or sixteen machines should be made, the layout for each four being repeated parallel to the first group of four, and the machines regrouped according to the product made. In this manner future extensions can be taken care of without interfering with the initial installation.

Handling the Product.—In placing the products machines, while economizing on space as much as possible, ample room must be provided to work and repair. Space must also be provided for the storage of pallets handy to the operator. Since getting the finished product away from the machines without delay is a very important part of the operation of a plant, sufficient room must be provided to allow an empty car or rack to be placed convenient to the machine operator before the filled one is removed. This can be done by arranging the track on which the cars come to and go from the machine in a continuous loop, the cars always moving in one direction. In this way, no time is lost by the operator while waiting for a full car to be removed before an empty one is set in place.

Another method that will reduce congestion around the machines and permit the operator to dispose quickly of the finished unit is to operate a belt conveyor from between two machines making the same unit to cars a short distance away. One offbearer can rapidly remove the units from the end of this belt to cars or racks. This method makes possible a continuous flow of blocks from the machines, as the operator can place more easily his product on a belt at a uniform height and position than he can place them on cars at different heights and positions.

For long pallets such as are used for brick, a conveyor can be used which is made of two parallel endless wire cables passing over rows of grooved wheels, each pair of wheels being keyed to its shaft to insure uniform speed of each cable.

Equipment.—In choosing equipment for a four-machine plant to produce three different concrete units, it is desirable to have one machine for each of the three products and a fourth as adaptable as possible for corners, fractions and specials, or to be reset to replace one of the other three in case of breakdown. If investment does not have to be too closely limited, it is often desirable to have more than one complete machine in addition to the number which it is intended to operate daily, these extra machines being already fitted for units commonly in demand, to facilitate a quick change when required. It is also especially desirable at all times

to keep on hand a complete stock of repair parts to avoid delays in shipment.

Grading Aggregates.—In planning the manufacture of concrete block, brick and tile, three different grades of aggregate will be required. For block, graded aggregate up to ¾ in. or 1 in. can be used; for light-weight tile up to ½ in. or ⅝ in.; and for brick, sand passing a ¼-in. to ⅜-in. screen.

For this purpose I would suggest that all aggregate coming into the plant be passed through a rotary screen separating the sand and gravel into two grades each, the fine sand through a screen having six to eight meshes per inch and the coarse through a screen having three to four meshes per inch. The fine gravel would pass through a ½-in. to ⅝-in. screen and the coarse through a ¾-in. to 1-in. screen. These screen sizes could, of course, be varied to suit the ideas of the manufacturer. Tailings, if produced in sufficient quantities, could be crushed and rescreened; otherwise they could be disposed of for other classes of work. These materials would be stored in bins with a unit measuring device, preferably of one or more cubic feet capacity to facilitate varying proportions.

In proportioning these materials for the concrete required for each of the different units, the mixes would be determined by the variations in the grading of the different sizes. If more fine sand was produced than was desirable in a proper mix, less of the fine sand than of the coarse could be used, the remainder being disposed of for other purposes. In this way a much better quality of sand would be obtained than would result from screening through one sand screen. This would be a convenient method of reducing the fines in an otherwise too fine sand. In order to get a fair amount of the fine sand through the screen it would probably be necessary to apply a pressure oil torch to the screen, or some form of vibration like that caused by a drop continually striking the screen. The difficulties of such an application need not be much of a hindrance.

While there would be some expense for installation and operation, it would soon pay for itself in the saving of cement that could be effected. In a recent case a manufacturer, by grading his aggregates, was able to produce 24 blocks per bag where he had been getting from 16 to 18 by his former methods.

When aggregate is bought, with several sources of supply available, it might be possible to choose them so that approximately the desired quantities of each size could be screened out. This method would greatly reduce the variations in the quality of the concrete, caused by wide variations which often occur in the materials as brought to the plant.

An Economical Method of Curing.—In curing products with steam it has been the custom to apply the steam over a period of twenty-four to thirty-six hours. This means that some degree of steam pressure must be maintained through the whole of this period. This, of course, requires operating the boiler during this time, with its consequent expense of fuel and labor. The object of this is to cure the product sufficiently so that it

can be safely carried to the yard and stacked. To accomplish the same result it is suggested that 2 per cent calcium chloride be added to the mixing water for all concrete except facing, that the product remain in the steam rooms during the night only, and be removed to the yard the following day. By this method the period of boiler operation would be reduced, the chloride would hasten the set sufficiently to compensate for the short time spent in the steam pressure, and the presence of the chloride in the product should tend to increase the absorption of moisture, thus aiding in the curing of the product and eliminating sprinkling the product in the yard.

As this method, without steam, has been much used to increase the turnover of cars and pallets, there seems to be every reason why this suggested use with steam should be a success.

Planning Operation.—In order to operate a plant profitably some schedule of production should be made. This is particularly true in a plant where sales will not warrant the operation of all machines every day. A suggested method of arriving at a schedule would be to estimate the probable annual demand for each product, determine approximately the number of days per season that the plant is expected to run, and proportion the number of days' operation for each machine, preferably per week, in order to produce this volume of product.

While, at times, such a plan might tend to run up somewhat large stocks, it would insure a well-cured product and eliminate extreme high and low points in production. It would also effect a saving in labor turnover, which is more costly in dollars and worry than most of us realize. In addition, it would improve the morale of the men, for the average man with a steady job is the more efficient workman.

Conclusion.—If anyone has tried any of these methods or has anything to add to the suggestions made, or any changes to suggest, I hope he will feel free to offer them. Since our profits depend on the economical operation of our plants, free discussion of the matter will surely be of much general benefit.

A RURAL CONCRETE PRODUCTS PLANT.

BY JOHN A. GOETZ.*

Much has been written, more said, many plans published and men and organizations have specialized in concrete products plants and layouts. It should certainly be an easy matter for a prospective manufacturer of concrete products to find a plan that will fit his site, and of equipment there is no scarcity of variety. This is especially true if the production is to be on a comparatively large scale. But, assume the

* Mattoon, Ill.

prospective "block maker" is establishing a plant in one of the smaller towns and rural communities! Say it's a city of 15,000 population with a maximum of perhaps that much more in the nearby towns and among farmers, keeping within the cruising radius of Ford trucks.

How would a well-built, thoroughly-equipped plant, with elevated bins, conveying machinery, measuring devices, power tampers, boilers, steam-curing kilns, mixers, lift trucks, and other niceties that go to complete such a plant, look in such a location? Such a plant, equipped with the smaller power machines would even then have a production capacity of perhaps eight or ten times that required.

Essentials.—We will all agree that in order to produce good concrete products proper mixing of the concrete, thorough tamping or ramming into the mold and curing under good conditions are essential. The mixing and tamping can hardly be done properly without power machinery, and a good building with some sort of heating plant is necessary for proper curing. All other machinery and devices, while desirable, are not necessary and good concrete products can be made without them.

Since there must be many localities with conditions similar to the ones mentioned, the writer will describe the layout of his own plant and the methods used.

Perhaps a goodly number of rural concrete products plants are a side-line to the contracting business, and this is one of them. Not so many of these plants operate during the winter months, but this one does, in fact most of the ordinary and staple products are made in January, February, and March. Thus we not only have a good stock on hand in early spring, but can also forget about making blocks for some time. The principal products are standard 8 x 8 x 16-in. concrete blocks, both plain and faced. Other products are caps, copings, sills, lintels, posts, culverts, vases, garden furniture, etc., the standard sizes and shapes of which are all made during the winter months.

Location and Description.—The plant is located on a lot 75 ft. wide and 100 ft. deep with a railroad siding at the rear and a paved street in front. The main room which houses the machines and also holds the products while curing is 26 x 50 ft., set along one side of the lot and adjacent to pebble and cement bins which are at one end next to the railroad siding. Sand and stone bins, heater room and entry are along one side, adjacent to the main room, with 8-ft. drives into the main room, and the office on the second floor. Products are removed from the front of the building towards the street, and also from one side. the machinery consists of a 7-ft. paddle mixer with a 5-hp. motor direct-connected to it, and one power tamper with elevator and feeder, belt-driven from a line shaft, driven by motor. The entire building is of concrete blocks, with a concrete roof over the cement bins, and a composition roof over the main building and aggregate bins. No hand truck being used, no floor is provided except around the tamping machine. The heating plant consists of a warm-air furnace, with provision for forced circulation by means of electric fans.

Materials and Handling Methods.—Materials used in the products consist of ⅝-in. to ¼-in. pebbles, ¼-in. torpedo sand, ¼-in. limestone screenings and portland cement. Pebbles are unloaded directly from cars to bin holding about 125 cu. yd. Cement is received in bulk and is also unloaded direct from car to bin, all by hand. Sand is hauled as needed from stockpile, as the bin holds but 20 cu. yd. Screenings are hauled direct from car to the bin which holds about 25 cu. yd. Bulk cement is used because the mixer does not have the capacity for a whole sack, for it is an easier matter to fill a bucket of correct size than to try to split a sack. There are other arguments in favor of bulk cement among which are lower cost, no money tied up in sacks, and—perhaps the most important—cement will not cake when loose in a bin as it does in sacks.

Proportions.—Concrete is mixed in the proportions of one part cement, one part ¼-in. limestone screenings, four parts ¼-in. torpedo sand and four parts ⅝-in. pebbles, With such proportions we get 22 to 23 blocks per cu. ft. of cement. Tests made on blocks show strength well above the standard requirements and the absorption is low.

Operation.—As the mixer is set so that all of the four materials can be reached in two or three steps, one man can mix the concrete for the one-block machine, and when making faced blocks can also prepare the facing mixture by hand. One man operates the tamping machine and another carries the blocks to the racks. On some classes of faced blocks we use two gas flames under the face plate, as it helps to keep the facing from sticking. This may not be necessary when using a dry concrete but we do not use dry concrete and with a minimum of facing material (we face as many as 100 blocks with one sack of cement and 1½ cu. ft. of sand) it helps a lot. As the block are removed from the machine they are set on 2 x 4's, the first of which are on the floor, with three additional tiers set one above the other and supported on chairs from the tiers below. In this way we get four tiers high, and by leaving passageways every third row we can reach all of the blocks to give them a thorough wetting the morning after making. Block are left in the building at least four days, and in order to conserve yard space are piled as much as 10 to 12 ft. high. To accomplish this we load the block on Ford trucks instead of wheelbarrows from which it is easy to pile them high. Ford trucks are also used for job delivery.

Production.—It may be of interest to know what such a plant can turn out. Our working day is nine hours and in that time we turn out an average of 500 faced block, or 750 plain block. By proper racking the curing room space will hold about 2,000 block, or three or four days' output. It takes one day to remove the block to the yard which leaves one or two days of the week for making repairs, hauling materials, making other products such as vases, garden furniture, posts, etc. If called upon this plant could easily turn out 100,000 block per year, which is twice as many as this particular locality uses.

Selling Some of 'Em.—As to the sales end of it—we have three lumber dealers here and through them about two-thirds of our entire output is marketed. It may be of interest to know what the arrangements are for such a deal. Most builders prefer to purchase all of their materials from one dealer and so when concrete block are specified, the dealer turns over the order to us. In turn we give the dealer a small discount from the established price, and also purchase cement in carload lots, as well as other materials, thereby keeping on the best possible terms and giving him no reason to try to sell other materials than concrete block when such are asked for. Our lumber dealers suggest the use of block to many prospective builders. Does any one know of a better plan to keep the material dealers in line?

Conclusion.—The writer has no competition from other concrete products plants and would be considered as "being in clover" but—clay products are competitive products and when low priced are used in many places where concrete block ought to be used. It may be said that with a good plant and some energy behind it, more concrete block should be used in such a territory as this. This is true, but it has not been very long ago when block were used only in the cheapest construction work, and even now there is much room for improvement.

This paper is not intended as a defense of "one horse" block plants, but since they are necessary, the best that can be done is to get them to produce good concrete products with the meager facilities that can be mustered. The American Concrete Institute and the Portland Cement Association have been and still are doing their utmost not only to produce good products, but also to cut the cost of production to the minimum.

UPS AND DOWNS IN PRODUCTS MANUFACTURE.

BY LACEY PEYTON.*

Many changes have taken place since I started looking into the possibilities of concrete products manufacture in 1901. At that time I was a clerk in a grocery store. In passing to and from my work I watched an Italian mason making cement blocks for a five-room residence. The procedure was to take about two wheelbarrows of sand to a box of cement containing 2 cu. ft., dump all on the board and cut through the mixture a few times, then sprinkle water lightly, being careful to get just the right amount—enough to consolidate the mass when tamped—put into a mold box and tamp or pat lightly, and then remove the form, place in a sunny place to dry and place in the wall tomorrow. This procedure looked too easy so I watched the results.

* Benton, Ill.

The house was finished and water filtered through the walls and ruined the plastering. Painting started (four coats of paint) but still there was moisture. Finally the walls were furred with 2 x 4-in. lumber, lathed and replastered. Then after the roof had been changed from a fire walled flat roof to a four-pitch hip roof with 24-in. projections all around, the place was dried out and became a home and is a good one today.

The next year one of the great concrete-arch bridges was under construction. I thought of the cement house. So I made up my mind to see some of this bridge work. Of course the water content was different from that used in the house, a fact which gave me a subject for thought. The bridge was finally finished and was pronounced a wonderful structure and as I passed this bridge to day (it is 6 miles north of Carbondale, Ill., on the Illinois Central) the above thoughts came back to me. Taking the two jobs into consideration, and seeing the results in each case I decided concrete products manufacture could be accomplished and I made up my mind that I would work it out as soon as I could get the necessary funds. Funds for the venture were not readily available, so I set to work out what I thought I must have in cash to get a business started with equipment I deemed necessary to carry on in a fair way.

After getting the necessary property and equipment I started my plant (all hand made, even to the mixing) using the aggregate and the amount of cement I thought necessary to produce a good block. I then found I would have to get 5¢ more than the lumber yards were asking for their cement blocks, which were of the cement-sand sun-cured type, so I started talking quality blocks, trying to make each batch better, larger aggregate, less cement, harder tamping and more blocks per man.

Heavy Tamping.—My idea of heavy tamping did not agree with that of the machinery producers of 1916, for I was advised by some of them that a 5- or 6-lb. tamp was the only one made, and if I used anything heavier I would spoil my product and waste a lot of energy. However, I made one which weighed 16 lb. and you should have seen the way this tamper with a little elbow grease brought the water to the form of the machine, producing a better block with less blows, and speeding up the output.

I believe that the weight of the tamper has much to do with the amount of water used to get certain results. With the heavier tampers one can use larger aggregates, less cement, less water and still get maximum results, leaving a clear water marking on all exposed parts of the product and sealing all minute openings.

In using a wet mixture of this type it is necessary to strip block off the face plate, and in the case of rock face or panel face plates it is necessary to use a fairly dry facing material (graded 3/16 in. down with a 1:3 mix will get good results), which brings up the argument that this costs extra money. It does, but it only costs about ½¢ per block for material and you can produce as many faced as plain blocks and get an additional 2¢ for the product.

My producing machine is of the automatic face down type and will make three block per minute of faced blocks at the same rate as the rough cast, as we call them.

Lets get back to the water-cement ratio: from a technical point I do not understand all I have read about it. However, the amount of water used in each batch is governed by the amount contained in the aggregates. When placed in the mixer more is added until we get a distinct water marking on the troweled side in the mixer which the average mixer man can soon learn. With a little experience he can come within a quart of the proper amount by careful study. To set a plant rule of 4 gal. to each 10-ft. batch will not work out in plant practice.

Many of us have been bothered with our block splitting through the webs when placed on the racks and in most cases you will hear the operator say the reason is too much water, while if you will use two or three quarts more you will eliminate the splitting and have a much better block.

Right here I would like to tell a little story which brings out my point clearly. John was at the mixer. The block were splitting, in part, on the racks. I suggested adding more water but John said less. We agreed to take three block out of this particular batch and diminish the water on the next one to his satisfaction. I said on the third batch I would use my judgment. At the end of 28 days these blocks were tested in a standard testing machine with the following results:

Blocks	No.	1	average	crushing	strength	164,000	lb.	3¼	gal. water.
"	"	2	"	"	"	176,000	"	4	" "
"	"	3	"	"	"	198,000	"	4¾	" "

Since the working out of this test I have had little trouble with the amount of water placed in each batch because I proved to John what the amount of water meant to the product.

Workability.—In putting Batch No. 1 through the machine you will have a nice easy working material. It will not stick or pull off on the head block.

Batch No. 2 is rather gummy, it sticks to the head block. The operator had to wipe the head block three times during the batch and it is too hard on the operator.

Batch No. 3 works just as well as No. 1, gives a good water marking all over the block; there is no extra trouble to the operator and a much better product for your trade is secured.

I start with coarse aggregate, using just as much as I possibly can, then add the fines until I get a fairly smooth product. The result is about this:

4½ cu. ft. ¼ in. to ⅝-in. gravel.
4½ cu. ft. ¼ in. coarse sand.
1½ cu. ft. ⅜ in. limestone screenings.
1 bag portland cement.

At an average of 22 blocks to the batch of this mix we get a crushing strength of 1,300 lb. per sq. in.

I note that there is quite a difference between this mixture and that of Mr. Warford and it seems there should be some discussion on this point.

Limestone Screenings.—I have found that the addition of some limestone screenings in my mixtures adds to its workability, slipping into place easier, sealing small pores and allowing more water to be used in the mixture. We used about 1½ cu. ft. to the batch.

No doubt a great many of you will feel that the above crushing strength is too great. It may be, but its a lot of satisfaction to see your walls, when settlement takes place, open in the joints rather than break through the vertical joints and core centers.

Cut Cast Stone.

By Henry P. Warner.*

The Onondaga Litholite Co. has been engaged in the manufacture of high-grade cast stone for over twenty years, which is about the length of time that cast stone has been sold. Looking back over that period, we can recognize some of the mistakes that have been made. It seems that perhaps the greatest mistake made was the education of the architect to the use of cast stone as a substitute for natural stone.

Sufficient emphasis has not been put on the fact that cast stone as a building material has special merits and characteristics which make it desirable for use in high-grade buildings. There has never been a serious attempt to create a demand for the material based upon its own merits.

Owing to the great variety of colors and textures that it is possible to obtain in cut cast stone, the manufacturer has been able to offer to the architects a material which now has come into wide use.

One of the greatest handicaps in the marketing of cut cast stone is the prejudice of some architects unfamiliar with the product who vision it as an imitation of natural stone and who, from lack of information, are not willing to believe that it can have merits of its own. Many architects feel that there is only one reason why they should consider the use of cast stone and that is on a price basis. We believe that there are too many manufacturers of cast stone today who are satisfied with this situation and who are, therefore, bending every effort to manufacture a cheap product, irrespective of quality. They believe that there is no demand or market for a cast stone which costs more than natural stone.

We, however, are glad to say that there is a rapidly increasing number of architects and engineers who are now using the product because of its merits and because they find that they need it and that it has qualities which make it valuable. The cast stone manufactured by using high-grade aggregates is always uniform. Buildings which have been erected during the last 20 years in which cast stone has been used, prove that as the years pass, the material improves with age and becomes more interesting.

It should be understood that a cubic foot of this material, manufactured from granite or hard marble and uniform throughout, is costly, and, were it not for the saving possible by molding cast stone to approximately the shapes required, it would be prohibitive in price. Cut cast stone has proven its desirability, which accounts for the rapidly increasing demand which manufacturers of this material are enjoying.

Cut cast stone has many advantages aside from varied colors and textures, it has a low absorption and a long life. It is also possible to

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produce very large units. These large units are reinforced with steel, and the architect is enabled to use the material in connection with a concrete or steel frame building in places and in sizes which would not be practical in natural stone. The architect and contractor, and other users of cast stone, perhaps have only a limited knowledge of what enters into its production such as is made by our company. Correct formulas are of great importance and ample facilities for manufacture are paramount necessities. Before actual casting, much work has to be done if cut cast stone is to be successfully produced. The manufacturer of this stone must have complete facilities for the building of details and setting plans.

The matter of service to the architect is of great importance. The drafting room must be well organized and handled with the idea that in every case, architects and contractors must be given every assistance. This means, of course, that the architects and contractors must be kept informed as to the progress of the work. Experts are employed who must be in a position to submit setting plans showing the practical ways of meeting any difficulties which may arise during the progress of the building. It is the business of the drafting department to check up each job as to what portion is covered by the contract; when the first delivery is required and when the general contractor will need the job shipped complete. The job is then assigned to an expert who goes over the information at hand and makes a list of the additional information necessary and the architect is promptly furnished with this list.

If the architect's drawings consist of a sufficient number of large-scale details, the lowest stonework or that portion to be required first is at once drawn up on standard size sheets. These drawings are generally made at ½-in. or ¾-in. scale, according to the character of the work.

Three sets of blueprints are sent through the contractor to the architect for approval and any further information required. On these prints in red crayon are noted the various dimensions, sections and full-size details required to complete the stonework shown on that particular sheet. A letter is also sent along calling for the same information. When these prints are returned all necessary corrections are made at once and again three sets are sent for final approval. At this time these sheets show all jointing of the stonework and the required plans and sections dimensioned complete for further use by the shop, the general contractor and the stone setter.

As soon as the prints are returned, finally approved, the necessary full-size shop details and the necessary section, bed and face templets are made. When these are completed, individual isometric drawings of all the different stones are made on shop cards on which are noted the stone letter, number required, shop details, templets, amount of steel required for reinforcing, location of anchor holes, setting hooks, color of stone, kind of finish required, cubic contents, and square feet of finish.

When the draftsman has completed the information on the cards, the whole job is then checked by another draftsman. As soon as the work is

checked and found to be correct, a record of all shop details and templets is sent to the shop office together with manufacturing lists of stone which are distributed to the various departments as a guide in getting the stone out in the correct order of shipment. Elaborate and complete daily records are kept in the shop office covering production in each department together with detail cost records which are turned over to the accounting department.

The demand for scheduled deliveries necessitates a large and flexible output. The conditions peculiar to each contract must be considered and the manufacture of the material must be planned so as to furnish the stone in the order required and when wanted for the proper progress of the building. This necessitates a production manager whose business it is to keep in touch with each contract and the progress being made and to co-operate with the contractors and architects so there may be no delay in the production of the stone.

The use of ornaments and ornamental stone in modern buildings has increased of late years and it has been found necessary to maintain a force of expert modellers and carvers at the plant. Experience has shown that a better co-operation with the architect and contractor can be maintained if the modellers employed by the manufacturers are allowed to make the models for this work at the factory. They can carry out the architect's wishes and ideas better than modellers who are not familiar with the process of manufacture. Where the models are made at the factory, photographs are submitted to the architect for approval, or, if he desires, he, or his representative, personally inspects the models after completion and approves them before casts are made.

The carving of the casts is usually done at the factory under the supervision of the modeller. However, it is possible to do the work at the job, if the architect so desires, giving the architect an opportunity to have the carving and completion of the work done under his supervision. This is, of course, a more costly procedure.

The pattern department employs two kinds of craftsmen—pattern makers and plaster workers. The superintendent decides the method to be used in building these patterns and assigns the work to the proper craft. The simple patterns are made entirely of wood. The more complicated patterns which are to be used for repeated molding, must be built sufficiently strong to stand the excessive strain of sand tamping, and, therefore, are also constructed of wood, and the undercuts are made possible by the use of loose pieces.

For economy in pattern making, where different lengths of the same section are required, we draw the pattern and use also a method which is called stopping off. For example, if the stone with the same section is required in lengths 4 ft., 5 ft., 6 ft., respectively, the pattern would be made 5 ft. long and for the extra length the pattern would be drawn the 1 ft. to the required length. For the short length the pattern would be stopped off thus marking in the sand the location for the end board.

When the complicated pattern is not to be used for repeated molding a combination of wood and plaster is used. The reason for this is that plaster for the complicated portions of the pattern is much cheaper than wood.

Sand molding requires a pattern that will draw easily out of the sand. Sand molding of the complicated profiles and undercuts are made possible by the use of piece patterns that may be taken apart while still in the sand mold. Some of these undercuts are cut in by stone cutters when it is more economical than to build a complete pattern.

In addition to the patterns just explained which are exclusively used for sand molding, we use also plaster and glue molds. These molds are developed from either clay or plaster models. The process used in making molds from clay models requires expert plaster workers. One method used is as follows:

A layer of paper is placed over the clay model, then a layer of clay about ½ in. to 1 in. thick on top of which is formed a plaster jacket. The plaster jacket is removed, the layer of clay and paper removed and then the plaster jacket is again placed on top of the model. A hole is provided in the back of the plaster jacket at the most practical point so that glue may be poured into the space between the model and plaster jacket which was formed by removing the ½-in. layer of clay. Care must be taken to have plenty of vents for the escape of air while the hot glue is being poured into the mold. The result of this process is a glue mold which can be used for from one to three casts after which the glue begins to rot and the detail is lost. The method most often used and when more than three casts are required is as follows:

From the clay model we get a plaster mold, then a glue model, from which the required number of plaster waste molds may be taken.

We have tried to show by these few examples that in sand molding the judgment used in making patterns, both from the practical and economical standpoint, must not be left to a novice but requires the knowledge and experience of an expert. Our patterns are made the exact size of the stone required except on the exposed or finished side of the stone for which we make the proper allowance as we recut all of these surfaces to expose the aggregate to give the effect desired.

The crushing, grading and mixing of this material require expensive crushers, rolls, pulverizers, conveyors, and elaborate screening facilities. The aggregate from which the stone is made must be of such a nature that it can be crushed and graded to at least three sizes, the largest of which will pass ⅜-in. mesh. The crushing machinery can be adjusted to produce the proper proportion of various sizes of aggregate required. Care should be taken not to produce too much fine aggregate. If too much of these fines are used in the mix, the resulting product is a stone having an unattractive appearance without good weathering qualities and having a high absorption. A mixture of this kind also requires the use of additional cement, which is reflected in the color of stone.

We use about a 1: 3½ mix composed of three different sizes of hard marble carefully graded and carefully weighed into the mixer and about 8 gal. of water to each bag of cement. We vary the quantities of the different sizes and use mineral colors to obtain textures and the color effects desired.

This is thoroughly mixed for about 4 min. in a stationary mixer before being transferred to the agitator which is a subsidiary mixer. The agitator is picked up by an electric crane traveling over the casting area, the material being continuously mixed until deposited in the sand molds, or as the case requires into plaster or glue molds. The mix is described by some as being of a soupy consistency. Our own homely description of the consistency would be that it must be a pasty mass that will flow or roll along in a sluggish manner in the mold at a grade of 1 in. to the lineal foot from the point where it is first poured.

Great care must be taken in pouring. Three carefully trained men attend on this pouring process: one at the valve of the agitator; a man properly to spread the mix to all corners of the mold; and the other to place reinforcement, anchors and lifting hooks in the proper position as may be required.

Colored tickets on wires are then placed on each casting. These tickets fully describe the stone, job number, letter, color, etc. The stones are left in the sand as long as necessary to assure safe handling and piling.

In our factory all exposed surfaces of the stone are recut. This is either done by hand or by machinery and at least ⅛ in. of the cast is removed, thus exposing the aggregate. Unless a material is first-class it is not possible to finish it by the process now in use in our factory. If the aggregate is properly graded, this process results in some very beautiful textures.

The stone we have been marketing is produced from very hard aggregates and is therefore costly to finish. We use steel rubbing beds, carborundum saws, carborundum planing machines, carborundum tooling machines, lathes and rubbing and polishing machines. In this department a compressor capable of producing air for the air hammers for 75 to 100 stone cutters is required.

The hand tools used by the stone cutters are identical with those used in the natural stone industry, for example: a fine crandall, coarse crandall, sharp-pointed chisels and pneumatic hammers equipped with chisels.

To apply the smooth rubbed planer or honed finish on flat stones, the rubbing bed or the carborundum planer is used. For molded surfaces the planers only are used. A large assortment of different size carborundum wheels is necessary varying in size from 24 in. in diameter to 6 in. For economy we place the different diameter wheels in gangs to cover the width of the molding. The wheels are then dressed to a zinc templet of the contour required. All the carborundum planers are specially-built machines to withstand the vibration of the very large wheels used in the cast stone industry. Some of the planers are equipped with a circular attachment for

planing flat, curved or molded surfaces having a radius from 2 to 80 ft. These planers are also equipped for economy with another attachment which enables us to plane to an entasis and irregular lines. Each planer is provided with an individual swinging crane to relieve the large traveling cranes from the duty of waiting on the planerman.

For the reason that the carborundum wheels are revolving at a terrific speed and on account of the rigidity of the arbor and the forced feed, it is necessary to keep these wheels sprayed with water under a pressure of 90 lb. allowing the wheels to cut freely and to prevent gumming and clogging.

We have used for a great many years carborundum tooling machines designed by our own engineers. These are motor driven and are so designed that the carborundum wheels will tool horizontal or curbed surfaces. These machines are also used to cut drips, raglets and for jointing off.

Hand finishes are produced with the same tools and by the same methods as in the natural stone industry, with the exception of the hammer-dressed finish. We use a wide chisel in a pneumatic hammer and therefore do not bruise the stone as we would if the old-fashioned patent hammer was used for this finish. The entire finishing department is equipped with a network of traveling cranes and overhead and ground tracks for quick handling of stone on and off bankers and to the loading platforms.

One of the most recent developments in the history of the business of our company is the production of a material which does not in any way resemble any particular natural stone. The surface of this stone is entirely recut. The use of rough tapestry brick has created a demand for rough surfaces on the stone trim. This finish is being put on while the stone is still green. The product is interesting and its cost compares favorably with the cost of finishes which of necessity have to be obtained by cutting or removing the surfaces of the cast, after the stone is hard.

Stone is manufactured in our plant in the order in which it is required at the building and is finished in the same order, a carload at a time. With thirty-five or forty contracts going through the factory at a time, it will be seen that a very careful check has to be kept on this material during the progress of its manufacture so that there may be no stones missing from the shipments when completed and to see that one carload shipment follows another in the quantities required and which vary on different class of buildings.

Cut cast stone is usually shipped, if the stone is heavy, in gondola cars where the stone is carefully packed in shavings. A complete list of the stone contained in the car is made and sent on with the bill-of-lading to the contractor so he may be able to check up the shipment and to know just what stone is contained in the car before it arrives at the job. In the winter time, these shipments are covered to protect them from the weather. If there are no facilities at the destination for handling large stone from

a gondola car, the stone is either shipped in box cars, or if very heavy and unwieldy, on flat cars.

The traffic department sees to the routing of these shipments and keeps track of them until they arrive at their destination. If the building is a large and important one, the service man meets the first shipment at destination to assist the contractor in handling the stone to the building. If the material is sold F. O. B. cars, destination, the handling and the setting are not in the control of the manufacturer but the contractors welcome any assistance we can give them, as it is to our mutual interest to have the stone handled carefully and placed in the building without breakage.

The service department with a number of service men must be on call to assist the contractor in handling and placing the stone in the building. These men must be practical stone cutters able to do any cutting or fitting at the job which may be necessary. After the stone has been placed in the building, the manufacturer's responsibility does not cease until the building is cleaned and pointed and accepted by the architect. This kind of service is essential and sometimes very costly. These service men must be expert in making any repairs necessary to the stone because of conditions prevalent during the construction of a modern building.

One of the greatest handicaps under which we labor is the lack of good competition. Our company would welcome and encourage competition from concerns having the proper facilities for the manufacture of a high-grade stone. We are convinced that there is a great field for this material, and the demand is so great that we find there are not enough concerns to take care of it. We have found it slow and expensive for so few to do the educational work with the architects, and we would greatly welcome the help of other responsible companies who would share the burden. Architects who contemplate using cast stone, continually ask the names of firms to be mentioned in specifications, who can provide proper competition, as well as a high-grade material. The number of these concerns is decidedly limited.

The Wet Cast Process of Manufacturing Concrete Stone.

By Robert F. Havlik.*

Processes.—There are three separate and distinct processes of making concrete stone. The most common one is known as the tamped process. In this process the stone is made of a relatively dry consistency. The concrete contains just enough moisture to hold the aggregates together but not so much that the stone would lose shape when removed from the molds. This is necessary as the stone is removed from the molds as soon as it is formed and the molds are used over and over again. If the stone were made wetter, the stone would sag when removed from the mold.

The second process is known as the pressed process. This permits of the use of slightly more water than the tamped process but requires expensive heavy machinery and is no longer in common use. The third process, with which this article will deal, is known as the wet cast process. In this process the stone is made of so wet a consistency that the stone must remain in the mold until the concrete sets. This usually takes from 12 to 24 hours and in some instances 48 hours.

Consistency.—In the wet process it is necessary to use an excess of water in order to produce stone with a satisfactory surface. The excess water is taken care of in two ways:

(1) When the sand mold process is used, the sand absorbs the surplus water; (2) when plaster, glue, wood or steel molds are used, the best results are obtained by making the backing concrete of a dryer consistency than the facing and this likewise absorbs the surplus water. In both cases the net amount of water actually used is greater than what is known as 100 per cent consistency. This, as explained above, is due to the fact that it is necessary to use extra water to produce a satisfactory surface.

Sand Molds.—The first wet cast stone placed on the market on a large scale was produced in sand molds. One reason for this is probably the fact that the stone sold was made in imitation of Bedford stone which has a smooth sandy surface and sand molds are especially adapted for such a surface. The originator of this process was Mr. Stevens of Harvey, Ill. Today some of the largest manufacturers use the sand mold process almost exclusively, and surface it with machinery similar to what is used in surfacing cut stone and marble.

Plaster Molds.—Plaster molds are used for the production of orna-

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mental and molded stone. In the sand mold process, the moldings are frequently formed with plaster inserts, which all manufacturers of wet cast stone use to a greater or lesser extent. Those who specialize in the sand mold process use them only for the production of such stone as cannot be produced in a plain sand mold or one with plaster inserts. Other manufacturers produce all their stone in plaster, glue, wood or steel molds. All molded ornamental pieces are produced in plaster molds except those that have heavy undercuts which are usually produced in glue molds.

Plaster molds are made in sections which are held together by a cap, though sometimes with clamps in addition to the caps. The parts of the mold are made with sufficient draft so that they can be removed from the stone without damaging it. Intricate pieces with heavy undercuts can be made with plaster piece molds but such are not so well adapted to the wet process as they are to the tamped process because they produce too many expensive seams in the finished product.

Glue Molds.—Ornamental pieces with undercuts are nearly always produced in glue molds because stone made thus has only a few seams in it as compared to that made in plaster molds. Glue molds consist of a plaster jacket and glue lining produced as follows: The model is set up on a slab or table and covered with a thin layer of clay. A plaster jacket is then formed over this layer of clay. When the plaster sets up the jacket and clay covering are removed. The jacket is then placed in the same position over the model and the intervening space which was occupied by the clay is filled with hot glue. This glue is allowed to cool over a period of about 24 hours. The jacket is then removed from the glue and the glue lining is cut into several sections so that the model can be removed. The glue lining is brushed with french chalk to remove all grease, washed with a solution of alum and water to harden the surface, and painted with white lead or a litharge paint. This paint allowed to set over night and the mold is then ready to be filled with concrete. It is best to leave the concrete in the glue mold for 48 hours as it usually takes that time to set to permit of easy handling.

In cold weather it is possible to secure upwards of four casts for each glue mold, but in warm weather only two or three. When the glue becomes too soft for further use, it is melted and mixed with some fresh glue and used over again. The expense of making stone in glue molds after the first mold is produced is relatively small. The glue used is the ordinary commercial casting glue sold by all big packing companies.

Wood Molds.—Plain stone, such as ordinary ashlar, window caps, belt courses, and water tables can be produced satisfactorily on a smooth flat cement floor in wood molds. For best results the joints between the sides of the molds and the cement floor should be painted with plaster of paris. The cement floor itself should be covered with a thick coating of grease. The best grease for this purpose is that produced by melting stearine wax and mixing it with kerosene in proportion of about 1 lb. of wax to 1 gal. of kerosene. The purpose of the grease is to prevent the stone from sticking

to the concrete floor. About one-third of the stone used in ordinary jobs can be produced in molds of this type.

As the floor of all steam curing rooms is the coldest part, stone produced in this manner does not harden as rapidly as that which is produced in molds set on tables. Consequently, it is usually necessary to leave the stone in these molds for 48 hours, although it can be handled in 12 hours, but with greater danger of damage.

Steel Molds.—On account of the high cost of making steel molds, they are used only for standard sizes of stone produced in large quantities. The use of steel molds has been practically limited in the past to the production of ordinary concrete blocks. Nearly all stone used in building construction must be made in sizes that suit the particular job. In consequence, it is practically impossible to standardize on the sizes of plain stone, so it is very doubtful whether steel molds will ever come into common use for the production of concrete stone substituted for terra cotta or bedford stone.

Curing.—In order to produce satisfactory concrete it must be cured properly. In order to cure properly the stone must be provided with heat and moisture. The temperature of the stone should be kept above 50 deg. for best results. Moisture should be kept within the stone by any suitable method such as covering stone with moist sand, sprinkling at frequent intervals with water or storing it in an atmosphere of live steam.

When stone is produced by the sand process it attains its initial curing in sand. As sand retains moisture readily the stone is kept moist constantly while it is allowed to remain in the sand, a method which produces satisfactory results.

Another method of curing is to sprinkle the stone at frequent intervals with water. This should be done as often as necessary to prevent the surface of the stone from drying out and over a period of at least three weeks if no other method of curing is used. The most satisfactory and rapid method of curing stone is by storing it in curing rooms kept filled with live steam at as high a temperature as is possible. Stone cured in this manner for 48 hours is as hard as stone, after three weeks or more, cured by sprinkling or other methods whereby the stone is merely kept moist at average room temperatures. In this method the steam is fed into the curing room through pipes that are perforated at frequent intervals or which have pipe connections through which the steam can escape into the curing room. Some manufacturers have the mistaken idea that live steam will draw moisture from the stone. To prevent this they immerse the steam pipes in a trough which is kept filled with water. Such a system will not produce nearly as good results as a system in which steam is fed direct from the boiler. Both heat and moisture are necessary to cure properly concrete stone, and if the steam is fed through a trough of water the room does not become nearly as hot as where the steam is fed directly. Stone cured in this manner is not much harder than that cured by sprinkling with water at ordinary room temperatures.

SURFACING.

Rubbed Surface.—Concrete stone can be surfaced in many ways. The best known methods are by rubbing the surface on rubbing beds, by tooling with chisels or by etching with acid. The rubbed surface is produced by surfacing the stone on the rubbing beds. A rubbing bed consists of a cast-iron bed from 8 to 14 ft. in diameter which revolves in a horizontal plane. The stone which is to be surfaced is placed on this bed. A constant stream of water is poured on this bed and this carries with it fine sand as an abrasive. This process is used for surfacing terrazzo or marble wainscotting. This type of surfacing is used most commonly on stone made in sand molds.

Ornamental stone has to be surfaced by other methods. Carved stone is surfaced with pneumatic chisels. Molded surfaces are surfaced with planers in which the abrasive consists of a carborundum wheel the shape of which corresponds to the shape of the molding on the stone. The stone is fed slowly against the carborundum wheel which revolves at a considerable speed. The surface produced in this manner is the same as that produced on the rubbing bed.

Tooled Surface.—The tooled surface is produced by cutting the surface of the stone with chisels operated by a pneumatic hammer. Any type of surface desired similar to what is used on natural stone can be produced in this manner. Only concrete cast by the wet process can be satisfactorily cut with pneumatic tools as tamped stone is not sufficiently hard in the early stages. This method of surfacing is used on carved ornaments.

Etched Surface.—An etched surface is produced with muriatic acid and water. When this method was first thought of the acid was applied with brushes and most users failed to secure satisfactory results. The process also proved rather expensive. In order to obtain best results by merely brushing with acid, concrete must be watched carefully and the acid applied at certain times. Otherwise the concrete becomes so hard that the acid will not etch it sufficiently. The brushing and acid method are rather expensive and are not used by any large-scale manufacturers.

The most satisfactory way of etching is to dip the entire stone in a tank containing muriatic acid and water. The solution usually consists of one part of acid to about three parts of water. Acid will remove the surface cement in from 5 to 15 min., depending on the age of the stone and the strength of the acid.

It is some times thought that etching concrete removes the details. Such is not the case if the stone is produced properly and due allowance is made for this method of surfacing. I have seen etched stone that retained all the original details desired by the architect. This method, in my opinion, is the most economical and satisfactory method of surfacing. There are some manufacturers who differ from this statement, particularly those who use the sand mold process and surface stone by rubbing; but my reason for this statement is that stone surfaced in this manner can be sold in competition with bedford stone and terra cotta, whereas rubbed stone

must be sold at a premium on account of the higher cost of manufacturing. In general, all three of the above methods produce satisfactory stone of pleasing appearance.

TEXTURES.

The texture for which there is the largest demand is that which imitates bedford stone. While it is easy to produce this texture, unfortunately it is one which checks easily. Manufacturers should make every possible effort to educate architects to use other textures which are less likely to check. This is a difficult matter because bedford stone has been so commonly used.

Terrazzo Textures.—Terrazzo textures are produced by the use of marble chips and black slag, or other colored aggregate. Those most commonly used are white marble chips and black copper slag. Stone produced in that finish is rubbed on rubbing beds and the stone is oftentimes called granite. Only wet cast stone can be surfaced satisfactorily in this manner. Being made by the wet process it is watertight. It is ground smooth and therefore remains clean longer than most other stone but the finish is costly and such stone invariably sells at higher prices than natural stone.

Granite Textures.—There is a difference of opinion about what constitutes granite textures. The terrazzo textures are frequently called granite textures, but I feel that the name should only be applied to such stone as is surfaced with genuine crushed granite. Crushed granite is not used in stone that is made with a rubbed surface because it is practically impossible to grind this satisfactorily on a rubbing bed. Some of the granite chips invariably will tear loose from the concrete and the resulting surface will have tiny pits in it. For this reason, when using genuine crushed granite for the surface it seems preferable to surface the stone by some other method, the most satisfactory of which appears to be etching. Such stone can be made quite smooth and the granite exposed at slight expense by the acid treatment. At a distance of 25 ft. or more such stone will look practically the same as natural granite which has been surfaced with chisels.

Special Textures.—Almost any special texture can be produced in cast stone that may be required for a given job by careful selection of suitable aggregates and if necessary, suitable coloring material. Quite often colored marble chips are used for certain color effects, as are also colored granites. A beautiful granite of light reddish tinge is produced by using crushed pink granite and white cement.

Tints.—There is a considerable demand for colored stone in subdued tints. Satisfactory stone of light shades of color can be produced by careful selection of the aggregates and coloring material. Buff, pink and green tints are the most popular. Cement colors only should be used for this purpose and only a sufficient quantity to tint the same. Principal color tones should be given by a careful selection of the aggregates. The cement must be colored, otherwise it is likely to have a dead appearance or if

white cement is used a chalky appearance which can only be corrected by the use of a proper amount of suitable coloring material.

Facings.—A granite facing can best be produced by crown point spar which is a crushed granite of light gray texture. This can be purchased in almost any desired size from the Crown Point Spar Co. of New York City. When used with white cement it produces a beautiful granite stone. Other granite surfaces can be produced by the use of coarse granite of various colors. Probably the most pleasing natural granite color is produced by the use of crushed red granite.

Marble is used extensively for facing. This material seems more suitable for this purpose than any material. It can be purchased in several sizes from several large companies. Black copper slag is used extensively by some manufacturers to be mixed with crushed white marble for the production of terrazzo surfaces or so-called granite surfaces. It makes a pleasing surface. It is also used with crushed marble for stone that is surfaced by the acid method.

Superior Qualities of Wet Cast Stone.—Engineers and architects are generally agreed that stone produced by the wet cast process is superior to that made by any other method, particularly tamped stone. It has not been as popular in actual use because of the fact that wet cast stone invariably sells at higher prices than tamped stone. Wet cast stone is harder than tamped stone because it contains more nearly the correct amount of water to produce the ideal consistency. Because of its hardness it is handled with much less damage. Being made by the wet process it is watertight. The waterproof qualities are independent of the use of any waterproofing compounds, and waterproofing qualities produced by waterproofing compounds will disappear usually within less than one year's time.

Because of its waterproofing qualities, wet cast stone is more stain resisting than tamped stone. If water cannot penetrate the surface of stone, all stains are bound to be on the surface only and therefore can easily be removed with diluted muriatic acid. The durability of stone is largely determined by its water-resisting qualities and because wet cast stone is watertight, it will endure forever.

We are prone to think that all concrete is everlasting, but it is a well-known fact that when tamped stone has been in a building for any period of years it becomes surface pitted. Copings, cornices and other members that are exposed to the weather will invariably show raw edges indicating that the cement has actually been eaten out or loosened from the aggregates. Stone made by the wet process shows no evidence of such disintegration. I feel free to make these statements because I have probably used tamped stone, pressed stone and wet cast stone in combination more extensively than any other individual in this country, and I have come to know quite well the weaknesses and advantages of the three processes. I have seen all three kinds used in the permanent buildings at Mooseheart, Ind., and where all three kinds were used under the same conditions and have been made of the same aggregates, it is readily to be seen that the

wet cast stone is cleaner in appearance, has a harder surface and is much whiter, and shows no evidence of disintegration. The Mooseheart buildings also show that stains cannot be removed from the tamped and pressed stone satisfactorily, but the wet cast stone can be cleaned to look as good as new merely by washing with muriatic acid and water.

The Soldiers' Memorial Stadium at Grant Park, Chicago, Ill., is probably the biggest single example of high-grade concrete stone used in one structure in this country. This was made by the Benedict Stone Co. of New York and Chicago. The stone for the first part of the structure was made in New York City and for the recent addition was made in Chicago. It is all made by the sand mold process and surfaced with machinery, most of it being rubbed on rubbing beds. Approximately $2,000,000 worth of cast stone was used in this structure. While the stone has darkened in appearance due to weathering conditions, it has weathered uniformly and has not turned black as does bedford stone or tamped stone.

Our company furnished the stone for the Olympian Fields Country Club, Chicago. Both tamped stone and wet cast stone were used on this job and both were made of identical material, the only difference being in the consistency of the concrete used in the two processes. Although this job is out in the country, the tamped stone has darkened much more than the wet cast stone.

My experience with tamped and wet cast stone finally caused our company to discontinue the manufacture of tamped stone. In our experience we have never had to apologize for the quality of any of our wet cast stone, whereas it is necessary to apologize for the quality of tamped stone in nearly all instances because no matter how well it is made, some parts of the job are likely to be of decidedly poorer quality than other parts.

Art Marble.

By I. L. Stearn.*

For many years, art marble has been the trade name of a concrete product manufactured by the Chicago Art Marble Co. Recent practice, however, has broadened the meaning of this term so that it is now understood to include all cast concrete products made with selected marble as aggregate. It is employed extensively for floor tile, trim, wainscotting, stair treads, and for ornamental purposes.

Art marble differs materially from other precast concrete products in the selection of aggregate, processes of manufacture, durability and appearance. While the components of other products may be similar in many cases to those used by us, the finish produced by various grinding and rubbing processes, which expose the wide range of color in the marble aggregates, distinguishes it particularly.

The success of art marble depends in no small degree upon the selection of the aggregate. Only carefully selected marble chips of known density, hardness and wearing quality are used with portland cement and highest quality lime-proof color pigments. By reason of its density, due to the process of manufacture and the physical properties of its components, art marble offers a maximum resistance to wear for such work as floors, stairs and ramps. A distinct advantage is that it permits the use of different colored tile with uniformly long wearing surfaces. Further, by reason of its texture which is obtained by the proper blending of aggregates, cement and color pigments, it possesses unusual artistic and ornamental properties.

A feature of art marble for stair tread and riser construction is the advantage of casting the tread and riser integrally and providing a cove at the intersection. Such a cove results in increased ease of cleaning. The design usually provides for rabbiting of the riser into the tread above, giving the appearance of monolithic construction after the stairway is completed.

Another feature of art marble is its wide adaptation for use in building construction. It is being used for such work as fascia, wainscot, partitions, shower receptors, window stools, bank screens, fountains, check desks, benches, etc.

Since this is a precast product, many designs of different forms and colors are possible of production. The architect has found in art marble one of the most flexible materials with which he deals. It permits the

* Chicago Art Marble Co.

display of originality and the execution of pleasing effects in building design. The prohibitive cost of natural marble often limits its use, especially when the designs are intricate. The usual practice with a natural stone is to use flat slabs which are united by bolts and screws. Such construction generally has inferior strength and lacks rigidity. It is most difficult to obtain curves and intricate shapes with such material. Art marble, on the other hand, lends itself admirably in this work, since it is easily cast in forms of irregular shapes. At the same time it is strengthened by reinforcing bars placed within the mass. Hence, it not only develops most desirable surface textures but possesses superior strength, durability and wearing qualities not common to natural marbles.

We are proud of our product and are continually striving to produce material with the highest merit for construction purposes. It has been the choice of many architects in important work. Some notable installations of art marble in Chicago are found in such buildings as the Union Terminal Station, the Palmer House, the Drake Hotel, the Union League Club and the Wrigley Building.

In our process of manufacture, we have found it essential to give considerable attention to each of the major steps, these being the selection of aggregates, the manipulation of materials, curing and surface treatment. In order to produce a product possessing density, strength and high wearing qualities, it is necessary to obtain the hardest marbles available.

In the past, extensive use has been made of the waste material from marble and granite works, but it was found that such aggregates were undesirable for our use. They were not uniform in quality and were not of the desired grades for developing wear and surface texture. Accordingly, we now obtain all our marble in spauls directly from foreign and domestic quarries. We are convinced that only quarry materials will provide the grade of aggregates required for proper execution. The softer marbles are extremely absorbent and are given to pitting and staining. We do not purchase crushed marble aggregate but do all our own crushing so as to obtain the desired sizes and shapes.

Marble spauls (or quarry waste) range from the size of walnuts to 15 or 20-lb. blocks. This material when received is placed into a jaw-tooth crusher that reduces the particles to sizes which can be conveniently handled by a gyratory crusher. As the material leaves the jaw-tooth crusher it falls upon an endless belt and is conveyed to the gyratory crusher for a second crushing. The gyratory crusher produces sharp, angular shaped particles and chips. Such material affords a better binding surface and helps to develop the desired texture.

From the gyratory crusher, the chipped marble passes into a rotary sieve with ¼-in. and ⅜-in. mesh. Bins are provided underneath the screens to receive the various sized particles.

For most of our work it is necessary to use color pigments with light-colored portland cement to develop delicate shades and textures. Recent demands have been for a combination of cement and aggregates wherein

the shade of the cement is similar to the predominant shade of the marble chips. The highest quality lime-proof color pigments are used for this work. The cement and pigment are thoroughly mixed in flour mixers until the color is uniform. The time of mixing varies from 45 min. up to 12 or 15 hours, dependent upon the colors desired.

Material measurements for our concrete work are, for the most part, by volume. In proportioning aggregates and cement used in our floor tile, stairs, wainscotting, etc., a 12-qt. pail is our unit of measure. The facing mixture consist of 2½ pails, or 1 sack, of portland cement including the color pigment and 4 pails of crushed marble together with a small amount of marble screenings and 4 to 4½ gal. of water. Half of the crushed marble is coarse material, passing a ⅜-in. screen and retained on a ¼-in. screen, the other half is fine material, passing a ¼-in. screen with dust removed.

The materials mentioned are varied for special installations such as anti-slip floor tile, stair treads, ramps and entrances to elevators, by making use of an aggregate known as alundum. This is a by-product obtained from electric furnaces and is a very hard gritty material. Fortunately, it can be obtained in 5 or 6 colors, and blends admirably with aggregates and cement. So accurately are we able to manipulate the materials that no marked difference is visible between sections having alundum and those without it.

Mixtures incorporating alundum aggregates have the following proportions: 2½ parts of portland cement including color pigment, 1 part of alundum, 3 parts crushed marble (preponderance of ⅜-in. particles), a small amount of marble dust and 4 to 4½ gal. water.

Floor tiles which require only one exposed face are backed up with a mixture of about the same proportions as the facing material. The backing material is a very dry concrete consisting of materials of the following proportions: 2½ parts of portland cement, 1 part of torpedo sand and 3 parts of limestone screenings, with only enough water to produce a mixture having a consistency of molding sand.

Our equipment for manufacturing floor tile consists of a large, specially-designed machine of our own construction, which alternately compresses and vibrates. This machine has a capacity of 1,500 to 1,800 tiles per day. Speed of the revolving table, the vibrating apparatus, press, etc., is geared down to satisfactory speed of production and to the capacity of workmen.

A tile mold consists of a steel frame and a steel plate which fits into the mold but which can be removed with the tile after it has been cast. Necessarily, the height of the steel frame should be equal to the desired thickness of the "rough" tile plus the thickness of the steel plate.

The process of manufacture consists of filling the lower half of the mold with the facing mixture, which has a wet consistency. As soon as this material is in place the mold is vibrated for several seconds. This reduces the voids to a minimum and forces a part of the excess mixing

water to the surface. The mold is then completely filled with the backing mixture which has a consistency of molding sand. This dry mixture absorbs a considerable part of the excess water from the facing mixture. A piece of tissue paper is placed over the filled mold which is transferred to the pressure machine. The pressure imposed upon the material within the mold is about 40 tons. This enormous pressure compresses the dry backing and wet facing together and produces a uniformly dense mixture with a low water-cement ratio.

After the compressed tile and mold leaves the pressure device the steel plate upon which the tile rests is pushed vertically from the steel frame by means of mechanically operated rods which are raised by cams.

Each tile is taken from the machine by means of the steel plate and placed upon a rack which, when filled, is trucked into the curing room. It is necessary to protect the tile by keeping it on the steel plate until the following day.

The curing process of art marble, as with other concrete work, is a very important step in the manufacture. It is essential that each piece be adequately cured before the surface treatment is begun. Floor tile are stored in the curing room for two weeks and kept moist by intermittent sprinkling at a temperature of 70 deg. F. After a rack full of tile has remained in the curing room for a week, it is hauled to the rubbing machines.

The tile is then ground on a large machine, consisting of a revolving steel table with a specially designed frame above, providing vertical rods which rest upon the tile. These rods are free to slide vertically through supports and are constructed so that weight discs can be attached to them. Usually a total weight of 75 lb. is used to hold the tile firmly against the grinding bed. Moist silica sand is used as the abrasive agent for grinding down the face.

Each tile requires about 5 min. for this preliminary grinding after which the edges are ground down on a smaller machine. Silica sand is again used as the abrasive agent.

Grinding leaves the surface more or less scratched due to the coarseness of the sand. Consequently, each tile is rubbed first with a No. 80 grit stone and then with a No. 200 grit stone. The No. 80, being the coarser stone, removes most of the scratches while the No. 200 tends to hone the surface.

After the tile have been ground and rubbed they are again stacked upon the rack and hauled to work benches where the surfaces are coated with a cement paste. The function of the paste is to fill any air voids which appear. Since the rubbing process during application of the paste tends to darken its shade, it is necessary to use a shade of filling slightly lighter than that used in the tile.

After a second week of curing the tile are rubbed down with the No. 80 and No. 200 grit stones, which completes the surface treatment and completes the manufacture.

The manufacturing processes involved in the production of our precast products, such as stair treads and risers, wainscotting and partitions, differ necessarily from those of floor tile manufacturing. The proportions used in the mixture are similar but the method of handling is different.

In general, these products are cast upon tables having concrete slab tops. Forms for flat sections such as wainscotting and partitions consist of wooden frames placed upon the table. For the stair treads and risers a mold is attached to the table so as to permit casting the tread and riser integrally. Such work as partitions with intersecting surfaces is handled in the same way. Special sections of irregular shapes are made in the standard molds used in products plants.

In the manufacture of a partition panel the form is filled about half full with a facing mixture having a wet consistency. This material is tamped with small wooden tampers, which consolidates the mass by wedging the larger particles of sharp, irregular marble chips, and brings a considerable part of the excess mixing water to the surface. A light trowelling produces a smooth surface upon which the reinforcing bars are placed. Bars are usually placed near the edges of the larger pieces and aid materially in giving additional strength.

After the reinforcing is in place a dry mixture of the same materials and proportions is sprinkled over the wet surface. The amount of material should be sufficient to dry up the surface. This is accomplished by further tamping after which the remainder of the mold is filled with a slightly drier mixture of the same proportions. This material is tamped and manipulated so as to remove as much excess mixing water as possible. Care must be exercised, necessarily, so as to prevent over-manipulation.

The amount of mixing water used is in excess of that required for setting of the cement, since it is necessary in the manipulation process of wedging the particles together. The removal of this excess water, however, by the method just described, assures a uniformly dense concrete which when adequately cured and properly surface treated produces a product of superior strength. Depending upon the season of the year and the temperature, our precast products are permitted to remain undisturbed in the molds for 36 to 48 hours. In some instances, it is advisable to increase the time so as to avoid injuring the product.

Removing the precast products from the molds and forms requires considerable care. Until the specimens have acquired some strength they must be protected from sudden shocks or mishandling. After removal from the molds the specimens are carried into a curing room and are handled in a manner similar to that described for tile.

After the two weeks' curing, specimens have attained considerable strength and are ready for the surface treatment process. Accordingly, the specimens are taken to the grinding tables or beds which are similar to those used for floor tile but necessarily larger. The smaller pieces, movable by hand, are stacked upon racks and hauled to the grinding tables on small trucks. The large, heavy pieces are handled by overhead mono-

rail with which they can be easily transported from curing room to the grinding and rubbing tables.

Grinding down of the surface and edges exposes the aggregate and helps to develop the surface texture desired. As with the floor tile, the surface of precast products is scratched more or less by the sand used as the abrasive. These are removed by rubbing or buffing machines on which coarse grit stone (No. 80) and fine grit stone (No. 200) can be used interchangeably. While machine grinding and rubbing are used as much as possible, our work requires considerable hand grinding and rubbing of irregular pieces.

After the surface scratches resulting from the grinding have been removed by the rubbing machines, the specimens are carried into a filling room. The surface is given a thin coating of cement paste having a slightly lighter shade than that originally used in the concrete. The specimen is then stored for about a week. This is followed by a second and final finishing with the coarse (No. 80) and fine (No. 200) grit stones which are attached to a swing arm on the buffing machine.

The process just described develops a perfect satin or eggshell finish. In general, the manufacture, curing and surface treatment require about four weeks. This length of time is necessary since extreme care must be exercised with each step of the manufacture.

We hear considerable about crazing of concrete surfaces, but have experienced no difficulty with crazing of art marble. Our contention is that the presence of crazing indicates poor or improper workmanship.

An important part of our organization is the drafting department where we maintain a competent staff for rendering service to the architect and builder. This department assists in the selection and design of the various types of units and classes of surfaces. The preparation of estimates and the taking of all measurements incidental to the design of special pieces are handled by this department. Here, also, originate all the detail drawings with complete shop orders and dimensions for the plant superintendent.

Our experience has shown the desirability of assuming the entire responsibility for the installation of art marble. The usual practice is, therefore, to contract for the material in place, although in some few cases, we have been justified in trusting the general contractor with the installation. Close supervision in preparation of the mortar beds, the grouting and placing and the final polishing are required in order that art marble will give the maximum amount of service.

Tests of Colors for Portland Cement Mortars.

By Raymond Wilson.*

Summary and Conclusions.

This is a progress report on an investigation of the effect of commercial pigments on the color, permanence of color and compressive strength of portland cement mortar.

Exposure **tests have been made on about 2,400 mortar discs colored with integral admixtures of 264 commercial pigments. Quantitative measurements of the color of the discs were made by the use of the Munsell Color System and from the changes in color due to exposure to weather some relations between chemical composition of the pigments and the color permanence of the mortars have been determined. Data on the effect of weather on the color during 6 mo. exposure are considered in this paper.**

All the pigments were subjected to exposure tests in 1 : 3 white cement mortars with 5 per cent and 10 per cent admixtures (by weight of cement) of the buffs and yellows, 1 per cent and 2 per cent of the carbon blacks and 3 per cent and 6 per cent of the other pigments. More intensive tests were made on 17 pigments of representative chemical composition; these pigments were used in studies of their effect on strength of mortar at various ages from 7 days to 5 years, and on color, color permanence and 28-day strength, as influenced by hydrated lime admixtures and curing conditions. Six pigments were selected as representative of the predominant chemical composition in the various hues for tests of the effects of mix and consistency of the mortar and quantity of pigment.

In the rather small quantities used in the greater portion of these tests most of the representative pigments which were of satisfactory permanence did not cause great variations in strength. Pigments containing calcium sulphate had the same general effect as admixtures of gypsum. Admixtures of from 1 per cent to 10 per cent of carbon black reduced the strength under all conditions of test though there were variations in the actual reductions obtained in groups of tests made at different times and in which there was some duplication.

Two samples of ultramarine blue gave results which appear not to be in complete accord; the tests were not parallel and were made at different times. This is probably because ultramarine blue is to some extent a chemically active admixture and different samples may be expected to have somewhat different effects.

Admixtures of 10 per cent and 20 per cent of ochre, red iron oxide, black iron oxide and chromium oxide resulted in somewhat increased 28-day strengths under the conditions used in one group of tests. Larger quan-

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tities of ochre, red iron oxide and chromium oxide caused reductions while only slight changes in strength were caused by 35 per cent or 50 per cent of black iron oxide.

The most important conclusions which may be drawn at present are:

1. Most of the pigments used in these tests, and which are probably representative of those now on the market, are of a satisfactory degree of permanence, so far as may be judged by 6 mo. exposure tests in 1:3 portland cement mortars.

2. The following types of pigments were suitable from the standpoint of permanence after 6 mo. exposure:

Buff, yellow, red: iron oxide pigments.
Green: chromium oxide.
Blue: ultramarine blue.
Brown: iron oxide, or iron and manganese oxide pigments.
Black: iron oxide, manganese dioxide, carbon black, bone ash and the so-called mineral blacks (coal, coke, and certain unidentified carbonaceous materials high in mineral matter).

3. Cadmium lithopone, zinc chromate and organic colors are not suitable for general use in portland cement mortar.

4. Among the pigments classed as of satisfactory permanence after 6 mo. exposure, no differences in degree of color permanence could be observed.

5. In cases where abrasion may result in exposure of the aggregate by removal of the coating of colored cement paste care should be taken to use aggregates of a neutral color or of a color which will harmonize with that of the pigment.

6. The ratio of pigment to cement is the most important factor affecting the color of mortar produced by a given pigment.

The effect of a given pigment on the color of a mortar as expressed in terms of the Munsell system is proportional to the logarithm of the quantity of the pigment, the *chroma* being directly, and the *value* inversely proportional.

7. The kind of cement and the mix and consistency of the mortar are factors of secondary importance affecting the color. Curing conditions and admixtures of hydrated lime make practically no difference in the color, other conditions being the same.

8. Under some conditions quantities of certain mineral pigments in excess of the 10 per cent generally recommended as a limit may be used without seriously impairing the strength. These tests are not broad enough in scope to serve as the basis of a general recommendation regarding the quantity of various pigments which may be safely added without serious reductions in strength.

9. In mortars containing the quantities of pigments usually used, the effects of water-cement ratio, mix, consistency and curing condition on the compressive strength are about the same as on mortar without pigment.

10. Iron oxide black is preferable to carbon black since equally dark colored mortar can be produced with the iron black with less reduction in the strength of the mortar.

INTRODUCTION.

The increasing importance of concrete for decorative and architectural purposes makes an investigation of commercial coloring materials for concrete of particular interest. This paper is presented as a preliminary report of tests now in progress in the research laboratory of the Portland Cement Association* for the purpose of determining:

(1) The tinting value of commercial pigments.
(2) Relation between chemical composition of pigments and color permanence of mortars exposed to weather.
(3) Effect of admixtures of representative pigments on compressive strength of mortars.
(4) Color permanence and compressive strength of portland cement mortars and pastes containing representative pigments, and tinting value of the pigments as affected by:
    (a) Kind of cement (white or gray portland).
    (b) Admixtures of hydrated lime.
    (c) Curing condition.
    (d) Mix.
    (e) Consistency.
    (f) Quantity of pigment.

Information regarding the tinting value of the pigments was obtained from observations of the color of the hardened mortar. Color permanence was tested by exposing the same mortar to the weather, duplicate specimens being kept indoors in a dark, dry place.

The Munsell color system was used to give a quantitative measure of the coloring effect of the pigments on mortar and the changes in the color of the mortar on exposure to weather. Exposure was started in March, and the last specimens were exposed in December, 1926. Although conclusions regarding the permanence of the pigments in mortars are necessarily tentative at this stage of the tests, and although there are some gaps and inconsistencies in the data, it is thought that the results already obtained are of sufficient interest to warrant publication.

MATERIALS.

Pigments.—In this report the coloring materials are called pigments to avoid confusion. Two hundred and sixty-four pigments, representing products of about 40 manufacturers and dealers, were used. A sufficiently complete chemical analysis to determine the general nature of the pigment was made on each sample.

* The co-operative arrangement for researches in the properties of concrete and concrete materials, which had been in effect between the Portland Cement Association and Lewis Institute since 1916, was discontinued on June 30, 1926. The researches covered in this report were started at Lewis Institute and are being continued by the same staff now reorganized as the research laboratory of the Portland Cement Association.

The pigments, classified according to hue, were of the following types of chemical composition:

Buff and Yellow

Iron oxide (ochre, sienna, artificial hydrated iron oxide)	53
Cadmium lithopone (cadmium sulphide and barium sulphate)	5
Zinc chromate	1
Organic color on an inert base	1
	60

Red

Iron oxide, natural or artificial (over 85 per cent Fe_2O_3)	47
Iron oxide (less than 85 per cent Fe_2O_3, remainder natural impurities)	27
Iron oxide, extended with calcium sulphate	9
Organic color on an inert base	2
	85

Green

Chromium oxide	21
Mixture of chromium oxide and ultramarine blue	1
	22

Blue

Ultramarine blue	15

Brown

Iron oxide	15
Iron and manganese oxides, umber	18
Iron oxide and carbon	8
	41

Black

Carbon black	19
Coal or coke	9
Iron oxide	5
Bone ash	5
Manganese dioxide	1
Unidentified	3
	42

Cements.—White and gray portland cements were used; the latter was a mixture of equal parts of 4 brands purchased in Chicago; unit weights, 94 lb. per cu. ft.

Hydrated lime.—High magnesia finishing lime; unit weight 29 lb. per cu. ft.

Aggregate.—Calcareous sand from Elgin, Ill., graded 0-No. 4, fineness modulus, 3.00; unit weight, 112 lb. per cu. ft.

TABLE I—OUTLINE OF TESTS OF COLORS FOR PORTLAND CEMENT MORTARS.

Exposure tests of mortar discs 4½ in. diameter, 1 in. thick.
Mix by volume.
Aggregate: Calcareous sand from Elgin, Ill., graded 0 to No. 4 (F.M.—3.00).
Consistency measured on 18-in. flow table.
Compression tests of 2 x 4-in. cylinders generally parallel to exposure tests.

Group	Pigment: Hue	Pigment: Number of Samples	Pigment: Per Cent by Weight of Cement	Kind of Cement	Hydrated Lime, per cent by Volume of Cement	Mix by Volume	Flow per cent	Curing Condition
EFFECT OF HYDRATED LIME AND KIND OF CEMENT								
1...	Buff	1	0, 5, 10	White or Gray	0, 10, 20	1:3	160	Moist 5 days; dry 2 days
	Yellow	3						
	Red	4						
	Green	2						
	Blue	1	0, 3, 6					
	Brown	2						
	Mineral black	3						
	Carbon black	2	0, 1, 2					
EFFECT OF CURING CONDITION								
2.....	Same as group 1			White	0	1:3	160	Dry 2 days Moist 2 days; dry 2 days
EFFECT OF MIX								
3....	Yellow	1	0, 5, 10	White	0	Neat, 1:2, 1:4, 1:5	160	Moist 5 days; dry 2 days
	Red	1						
	Green	1						
	Blue	1	0, 3, 6					
	Iron black	1						
	Carbon black	1	0, 1, 2					
EFFECT OF CONSISTENCY								
4.....	Same as group 3			White	0	1:3	105, 120, 160, 215	Moist 5 days; dry 2 days
EFFECT OF QUANTITY OF PIGMENT								
5....	Yellow	1	0, 0.5, 1 2.5, 5, 10 20, 35, 50	White or Gray	0	1:3	160	Moist 5 days; dry 2 days
	Red	1						
	Green	1						
	Blue	1						
	Iron black	1						
	Carbon black	1	0, 0.1, 0.2 0.5, 1, 2 4, 7, 10					

OUTLINE OF TESTS.

In general, the medium of tests was a cement mortar of a 1:3 mix by volume, moist cured 5 days and air-dried 2 days before tests of tinting value of the pigment and color permanence of the mortar were begun.

An abridged outline is given as Table I, covering tests of the effect of *representative* pigments on the color and the permanence of the color of mortars as affected by quantity of pigment, mix, consistency, curing condition, kind of cement and admixtures of hydrated lime.

For these tests 17 pigments were selected on the basis of their chemical composition, almost every type of pigment among the 264 samples received being represented. These pigments are described in Table II. In

TABLE II—DESCRIPTION OF PIGMENTS.

Description of representative pigments based on chemical analysis.
Pigments marked * selected as representative of predominant chemical composition. Used in Groups 3 4, 5 (see Table I).

Pigment		Description
Hue	Sample	
Buff	8297	Ochre
Yellow	8252*	Ochre
	8245	Cadmium lithopone
Red	8240*	Iron oxide—97% Fe_2O_3
	8251	Iron oxide—41% Fe_2O_3, 48% $CaSO_4$
	8257	Iron oxide—40% Fe_2O_3, 43% $CaSO_4$
	8237	Organic color on barium sulfate base
Green	8253*	Chromium oxide
	8238	Chromium oxide and ultramarine blue
Blue	8239	Ultramarine blue
	8619*	Ultramarine blue
Brown	8259	Iron oxide and carbon
	8254	Iron oxide
Black	8258	Coal
	8242*	Iron oxide
	8303	Manganese dioxide
	8298	Bone ash
	8241*	Carbon black

the tests of the effect of mix, consistency and quantity of pigment, only six samples were used, these being representative of the predominant chemical composition.

In the group of tests of the effect of quantity of pigment, admixtures varying from 0.5 per cent to 50 per cent of the weight of the cement were used (0.1 per cent to 10 per cent of the carbon black). In all other tests two quantities of each pigment were added depending on assumed tinting values of the pigments of various hues. Admixtures of 5 per cent and 10 per cent of the yellow and buff pigments, 1 per cent and 2 per cent of the carbon blacks, 3 per cent and 6 per cent of all others were used.

In one group of strength tests 2 x 4-in. cylinders were made for test at ages of 7 and 28 days, 3 mo. and 1, 2 and 5 years. Other strength tests

were made at 28 days only and were closely parallel to the exposure tests given in the outline.

Exposure tests of a routine nature were made on the remainder of the pigments as admixtures in 1:3 white cement mortar. A few general observations from the results of these tests are discussed but no detailed data are included.

EXPOSURE TESTS.

In order to avoid possible non-uniformity in mixing the fine materials, the pigment, cement and lime in each batch were tumbled with a charge of pebbles in a 2-qt. glass jar revolved in a Deval abrasion machine for 20 min. The cement-pigment mixture, separated from the pebbles by means of a sieve, was mixed dry with the sand, sufficient water was added to give the desired workability, and the batch was thoroughly kneaded by hand. Workability or consistency of each batch was measured by means of the flow test, using an 18-in. flow table in accordance with the usual practice. Mortar discs were molded in forms consisting of 4½-in. circles of 1 x 1/16-in. strap iron having the ends bent outward along a radius and held together by a U-shaped spring clip. The forms were placed on plate glass or machined steel plates covered with waxed paper. The discs were molded with only enough trowelling to fill the form completely and to give a smooth surface; after the mortar had stiffened somewhat it was lightly brushed with a damp paint-brush to avoid a glassy surface. Two discs were made from each batch, kept overnight under wet burlap, removed from the forms, kept 4 days in the moist room protected from falling spray, air-dried 2 days in the laboratory, and the color determined as described later. One disc from each batch was placed in a dark, dry place in the laboratory and the other exposed on the roof to the weather and sun, with its face sloping to the south at an angle of about 40 degrees with the horizontal. Color determinations were made after exposure for 14 and 28 days, 2, 3, 4 and 6 mo., and will be continued at later ages. The exposed discs were air-dried one day before color determinations were made.

COLOR MEASUREMENT.

The Munsell Notation* was used for describing the color of the mortars. By this system a color sensation is described in terms of three independent qualities, *hue, value and chroma. Hue* is the property of a color which distinguishes red from blue, yellow, etc. In the Munsell system there are five simple hues, namely, red, yellow, green, blue and purple, and five compound hues intermediate between the simple ones. *Value* describes the light-reflecting quality or the lightness or darkness of a color. *Chroma* is the intensity of the color and may be considered as its departure from a gray or neutral of equal value.

A graphic presentation of these properties is essential to an understanding of their relations (see Fig. 1). For this purpose the 10 simple

* For more complete information on the Munsell Color System, see various publications of the Munsell Color Co., Baltimore, Md., particularly "A Color Notation" by A. H. Munsell, and the "Atlas of the Munsell Color System."

and compound *hues* designated by their initials as *R*, *YR*, *Y*, *GY*, *G*, *BG*, *B*, *PB*, *P* and *RP*, are represented as equally spaced points on the circumference of a horizontal circle in the order in which they occur in the spectrum, with purple, a natural step between red and blue, closing the circle. The exact portions of the spectrum taken as the 10 hues were so chosen that hues on opposite ends of diameters of the hue circle are true complements and the differences between adjacent hues are equal. Intermediate hues are designated by numbered steps as shown in Fig. 1. A hue midway between yellow-red and yellow would be described as *10YR* or *1Y*.

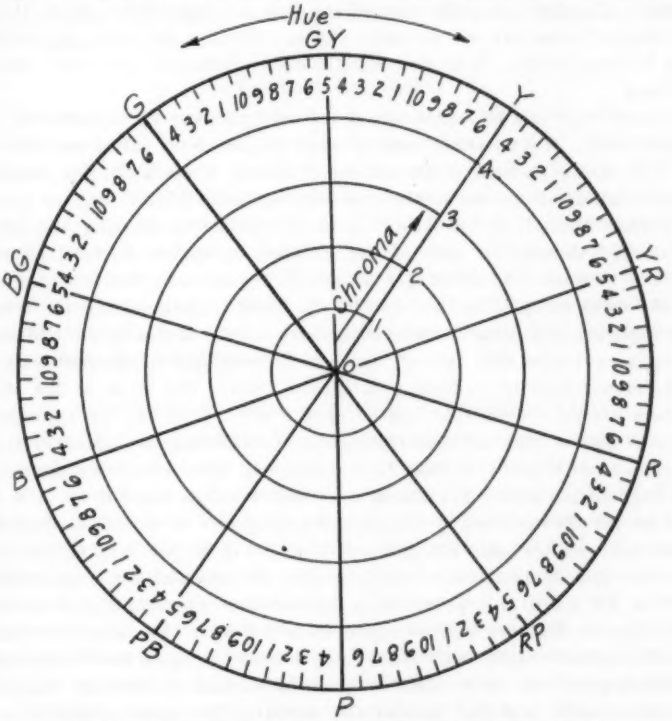


FIG. 1.—HUE AND CHROMA IN MUNSELL COLOR SYSTEM.
The value scale is perpendicular to the plane of the figure.

Value is represented as steps along a vertical axis, black at the bottom and white at the top, through the hue circle perpendicular to the plane of Fig. 1. Pure black and pure white, both practically unattainable, are value 0 and 10, respectively. A series of grays regularly graduated from 1 to 9 form the value scale, the basis of graduation being the proportion of incident light which is reflected. The grays along the axis are known as *neutral* gray and are designated by the symbol *N*. In the same way a value scale for any hue is represented by a line parallel to the neutral axis in the plane of the hue.

Chroma is represented as a series of radial steps from the axis. The somewhat arbitrarily fixed steps of chroma are so chosen that they are of equal degree in all parts of the color solid.

To combine these graphic portrayals of the three properties of color, we may think of an irregular solid with a vertical axis representing the grays. Vertical planes radiating at equal angles from the axis represent the 10 hues; horizontal planes equally spaced represent the 9 steps of value; the distance from the axis represents the chroma. All colors in the same vertical plane through the axis are of the same hue (or its complement if the plane on both sides of the axis is considered); all in the same horizontal plane are of the same value, reflecting the same proportion of the incident light; all at the same distance from the axis have the same chroma.

Any color can be completely described by giving its co-ordinates in the color solid. The notation usually used to give hue, value and chroma is *H V/C*, using decimal steps for any property if needed. For example, a certain shade of maroon might be described as *6R* 2.5/3.5.

The "Munsell Color Atlas" is a set of charts showing the steps of value and chroma for each of the 10 hues, by means of small accurately colored plaques. A set of 371 standard 4½-in. color discs corresponding to the color samples in the "Atlas" was used in determining the colors of the exposure specimens. These discs have a hole at the center and are slit along a radius so that two of them can be overlapped, mounted on a shaft and rotated rapidly to form a composite color. The steps of the Munsell system are so spaced that the composite color produced by spinning two discs is on the line between the points representing the colors of the two discs, and at a point on that line determined by the ratio of the areas of the exposed portions. For example, if the two discs are *YR* 4/5 and *B* 6/5 and 50 per cent of each is exposed, the composite color will be *N* 5/0, the point midway between *YR* 4/5 and *B* 6/5. If 70 per cent of the yellow-red disc and 30 per cent of the blue disc are exposed the composite color will be *YR* 4.6/2, this point being between the colors of the discs and 30 per cent the distance between them from *YR* 4/5. The color of a mortar which was not exactly matched by any one of the discs was determined by combining two or three suitable discs in various proportions until their composite color matched that of the mortar. The three properties of the color of the specimen were calculated from the colors of the discs and their exposed areas, the *value* by taking a weighted numerical average, and the *hue* and *chroma* graphically.

In order to avoid the effect of minor variations in the color and texture, the mortar discs were placed on a small wooden table mounted on a vertical shaft and rotated at a sufficient speed to give them a uniform and reasonably smooth appearance. The standard color discs were rotated on the shaft of a small electric fan motor. The surfaces of the mortar and the standard discs were in about the same plane and about 2 in. apart at their nearest points. The apparatus was set up under a light equipped

with a blue filter to give light approximating sunlight which would be constant throughout the tests.

Great care was necessary to prevent color changes due to accumulation of dirt on the standard discs. They were frequently checked against the corresponding color cards in the Munsell "Atlas" and replaced whenever necessary.

The same operator was usually able to check his determination of value or chroma within three and of hue within two decimal steps. Different operators were generally able to check each other within five decimal steps of value and chroma and within about the same limits as the individual operator on hue.

COMPRESSION TESTS.

The mortar for compression tests was prepared in the same manner as that used for exposure specimens and when feasible exposure and strength specimens were made from the same batch. The mortar was placed in the 2 x 4-in. molds in three layers and each layer rodded 25 times with a ¼-in. bullet-pointed steel rod. The top of the specimen was trowelled immediately after molding and covered with a glass plate; four to six hours later it was capped with retempered neat cement paste and covered with plate glass. About 18 hours after molding the cylinders were removed from the forms and stored under water until test, except for one group which was stored in air.

DISCUSSION OF COLOR AND COLOR PERMANENCE TESTS.

General.—The use of the Munsell system gives a quantitative measure of the change in the color of mortars on exposure to weather.

The most important change in the color of a mortar due to fading of the pigment caused by weathering is a progressive weakening of the *chroma*. Accumulation of dirt and soot results in a lowering of the *value* and no doubt also plays some part in causing the chroma to become weaker. The mortar discs were not washed after exposure and before their color was determined, but when the one-year inspection is made it is expected to determine the color both before and after washing. Just after a hard rain the value appeared to be somewhat higher and the chroma slightly stronger.

In some cases comparatively large changes in hue and chroma were caused by exposure of the aggregate, resulting in the mortar assuming a mass color which is a composite of the colors of the aggregate and the cement paste and pigment. By the method of testing used, the composite color of the discs was determined rather than that of the portion of the mortar discs where the aggregate was not exposed.

The average color of the aggregate was 1Y 5.1/2.3. The character of the color changes in mortars of various hues due to exposure of this aggregate will be understood by a reference to Fig. 1. Green and red mortars undergo changes in hue and chroma in a direction toward the hue and chroma of the aggregate; yellow mortars are not greatly affected in either

hue or chroma; the mortars containing ultramarine blue which are purple-blue do not change greatly in hue but exhibit more marked changes in chroma, since the color of the aggregate is the complement of that of the pigment.

The brushing of the discs tended to produce a very thin film of cement over the aggregate, particularly if the mortar was brushed while it was still too wet or if brushing was delayed too long. It is probable that the sloping position of the disc caused a more extensive exposure of the aggregate than would ordinarily occur. While excessive apparent fading due to this cause may not be particularly important in practice, it seems that permanently agreeable color effects will most surely be obtained by the use of an aggregate whose color harmonizes with that of the colored cement paste.

Color determinations up to 6 mo. show comparatively little change in the chroma of most specimens after 2 mo., and in cases where failure of pigment occurred within 6 mo., it was complete or definitely indicated at 2 mo.

Aside from the pigments which failed completely, no differences in the degree of permanence of pigments developed during 6 mo. exposure. Weakening of the chroma was of general occurrence during the early stages of exposure. Pigments which did not fade completely during the first 2 or 3 mo. of exposure and did not show considerable color changes thereafter were considered as satisfactory from the standpoint of permanence, so far as could be judged from a 6 mo. exposure period.

The mortars containing the pigments of representative chemical composition showed color changes on exposure to weather which were typical of those observed with other pigments tested. The character of these changes is indicated in Table III.

The cadmium lithopone (8245) had faded to a faint yellow during 3 mo. exposure. After 28 days the mortar colored with an organic red (8237) had only traces of its original color. Similar fading was noted in mortars containing the four other samples of cadmium lithopones, the other organic red and the organic yellow.

The failure of these pigments was evidently due to the action of weather, probably the fading effect of sunlight, and not to any chemical reaction with the cement. The surface of the specimen had assumed a color comparable to that of uncolored mortar, while below the surface the color was not greatly changed, nor had the discs kept in the laboratory in the dark undergone any change.

One pigment, a zinc chromate, failed because of chemical reaction with the cement. A 5 per cent admixture of this pigment caused quick stiffening of the mortar and the color faded almost completely within 24 hours; the mortar containing a 10 per cent admixture solidified before mixing was complete.

The yellow pigments which failed are evidently the result of efforts to produce a satisfactory lemon yellow pigment, as the mortars colored

TABLE III—EFFECT OF EXPOSURE TO WEATHER ON COLOR OF MORTAR.

Exposure tests on mortar discs 4½ in. diameter, 1 in. thick.
Mix—1:3 by volume; white portland cement.
Consistency to give flow of about 160 on 18-in. flow table.
Aggregate: calcareous sand from Elgin, Ill., graded 0 to No. 4 (F.M.—3.00).
Pigments: per cent by weight of cement.
Exposure started when specimens were 7 days old, after 5 days moist curing and 2 days air-drying. Exposed specimens air dried 1 day before color determination.
Color expressed in terms of Munsell notation: Hue (H), Value (V), Chroma (C).
See Table II for description of pigments.

Pigment			Color of Specimen											
			Before Exposure			After 28 Days Exposure			After 3 Months Exposure			After 6 Months Exposure		
Hue	Sample	Per Cent	H	V	C	H	V	C	H	V	C	H	V	C
		0	5YR	8.7	0.3	5YR	7.2	0.2	5YR	6.7	0.3	5YR	5.0	0.3
Buff	8297	5	2Y	8.0	2.9	1Y	7.4	1.9	3Y	6.4	1.6	10YR	5.5	1.6
		10	1Y	7.7	4.2	1Y	7.0	3.3	1Y	6.4	2.8	10YR	5.5	2.4
Yellow	8252	5	2Y	8.0	3.8	1Y	6.7	2.2	1Y	6.5	1.9	1Y	5.1	1.9
		10	10YR	7.3	4.6	1Y	6.7	2.4	1Y	6.5	2.4	1Y	5.0	2.3
	8245	5	9Y	8.3	3.8	9Y	7.3	1.4	5Y	6.7	0.7	2Y	5.4	1.0
		10	8Y	8.2	5.4	10Y	7.5	4.1	7Y	7.0	1.1	5Y	5.7	0.7
Red	8240	3	6R	5.4	5.4	6R	5.1	4.9	7R	4.8	3.3	8R	4.1	3.0
		6	6R	4.5	5.8	7R	4.8	4.5	8R	4.7	3.8	7R	4.0	3.8
	8251	3	6R	6.7	4.7	5R	6.5	4.0	8R	5.7	2.8	7R	4.5	2.4
		6	7R	6.1	5.5	7R	5.4	4.4	6R	5.0	3.2	7R	4.2	2.6
	8257	3	2R	6.4	1.9	10RP	5.7	1.5	3R	5.4	1.4	5R	4.2	0.8
		6	3R	6.0	2.6	1R	5.2	2.1	10RP	5.0	1.9	5R	4.0	1.0
	8237	3	2R	6.0	4.4	5R	7.3	0.3	5R	6.4	0.4	5YR	5.2	0.2
		6	2R	5.6	5.6	5R	7.0	0.5	5R	6.5	0.3	5YR	5.2	0.2
Green	8253	3	1G	6.6	1.9	9GY	6.0	1.9	10GY	5.8	1.7	5GY	4.1	1.1
		6	10GY	6.3	2.9	9GY	5.5	2.4	7GY	5.0	2.0	5GY	4.0	1.2
	8238	3	9GY	6.5	1.0	7GY	6.0	1.0	1G	5.4	0.7	5GY	4.4	0.6
		6	10GY	5.5	1.3	5GY	5.2	1.0	6GY	5.0	1.0	1GY	4.0	1.0
Blue	8239	3	5PB	7.0	3.5	4PB	6.0	2.8	4PB	5.6	2.4	5PB	4.2	1.0
		6	6PB	5.4	5.9	4PB	5.1	3.8	5PB	5.0	4.0	5PB	4.0	1.7
Brown	8259	3	5R	6.2	1.0	1R	5.4	1.0	5R	5.3	0.7	5R	4.0	0.8
		6	3R	5.4	1.4	3R	5.1	1.1	5R	4.7	0.6	5R	3.8	0.8
	8254	3	2YR	5.5	1.6	1YR	5.0	1.3	1YR	4.6	1.0	3YR	4.1	1.1
		6	2YR	4.5	1.8	2YR	4.4	1.3	2YR	4.3	1.0	1YR	3.8	1.4
Black	8258	3	N	6.4	0.0	N	6.0	0.0	N	5.6	0.0	N	4.7	0.0
		6	5PB	5.4	0.4	N	5.2	0.0	N	4.9	0.0	N	4.0	0.0
	8242	3	5PB	4.8	0.2	N	4.5	0.0	N	4.1	0.0	N	3.7	0.0
		6	5PB	3.8	0.2	N	4.0	0.0	N	3.8	0.0	N	3.2	0.0
	8303	3	5PB	7.2	0.2	N	6.7	0.0	N	6.1	0.0	5YR	5.3	0.4
		6	N	6.0	0.0	N	5.6	0.0	N	5.3	0.0	5YR	4.8	0.4
	8298	1	N	6 ·	0.0	N	6.1	0.0	N	5.7	0.0	5YR	5.0	0.3
		2	N		0.0	N	5.5	0.0	N	5.2	0.0	N	4.3	0.0
	8241	1	N	4 1	0.0	N	4.5	0.0	N	4.4	0.0	N	3.3	0.0
		2	5PB	3.8	0.3	N	4.0	0.0	N	3.7	0.0	N	3.1	0.0

with them are yellow to green-yellow in hue. No suitable pigment of this hue was received for these tests. Further comment on the hues produced by various pigments and their relative tinting values is not attempted in this paper, except as affected by factors such as mix, consistency, etc.

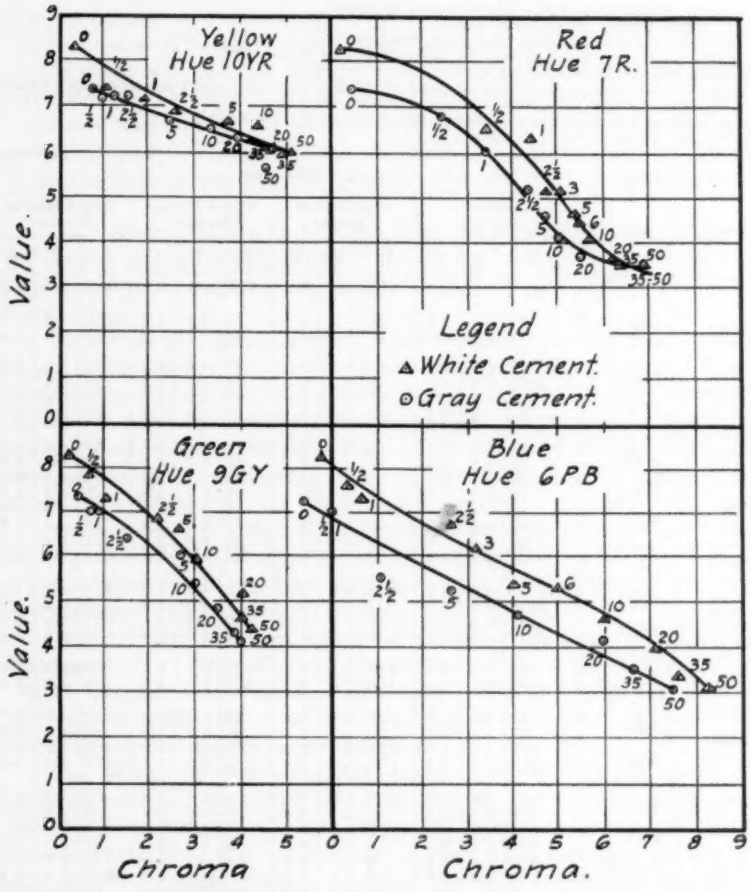


FIG. 2.—VALUE AND CHROMA OF 1 : 3 MORTARS CONTAINING VARIOUS AMOUNTS OF REPPESENTATIVE PIGMENTS.

Numbers opposite plotted points indicate per cent of pigment by weight of cement. Data from Table IV. Compare Fig. 3.

Effect of Quantity of Pigment and Kind of Cement.—The effect of quantity of pigment on the color of the mortar is shown for six representative pigments in Table IV. There was little variation in the hue produced by various amounts of the same pigment, even with the two different

TABLE IV—EFFECT OF QUANTITY OF PIGMENT ON COLOR OF MORTAR.

Color determinations on mortar discs 4½ in. diameter, 1 in. thick.
Age 7 days, 5 days moist curing and 2 days air drying.
Mix—1:3 by volume.
Consistency to give flow of about 160 on 18-in. flow table.
Aggregate: calcareous sand from Elgin, Ill., graded 0 to No. 4 (F.M. 3.00).
Pigments: per cent by weight of cement.
Color expressed in terms of Munsell notation: Hue (H), Value (V), Chroma (C).
Portland cements: white or gray, the latter a mixture of 4 brands purchased in Chicago.
See Table II for description of pigments.

Pigment, per cent	Yellow (8252)						Pigment, per cent	Red (8240)					
	White Cement			Gray Cement				White Cement			Gray Cement		
	H	V	C	H	V	C		H	V	C	H	V	C
10	5Y	8.3	0.3	5Y	7.3	0.7	0	5Y	8.3	0.3	5Y	7.3	0.7
20.5	1Y	7.4	1.0	3Y	7.2	0.9	0.5	7R	6.5	3.4	9R	6.8	2.4
51.0	1Y	7.2	1.8	2Y	7.2	1.2	1.0	8R	6.3	4.4	8R	6.1	3.3
32.5	10YR	6.9	2.5	1Y	7.2	1.5	2.5	8R	5.2	4.7	7R	5.2	4.3
5.0	8YR	6.7	3.7	1Y	6.7	2.4	5.0	7R	4.7	5.4	7R	4.7	4.7
0.0	9YR	6.6	4.3	1Y	6.5	3.3	10.0	7R	4.1	5.7	7R	4.2	5.0
0.0	9YR	6.2	4.7	10YR	6.3	3.9	20.0	6R	3.6	6.2	7R	3.7	5.5
5.0	9YR	6.0	4.9	9YR	6.1	4.5	35.0	6R	3.5	6.4	7R	3.5	6.9
0.0	8YR	6.0	5.0	9YR	5.7	4.5	50.0	6R	3.5	6.9	6R	3.5	6.9

	Green (8253)							Blue (8619)					
	White Cement			Gray Cement				White Cement			Gray Cement		
	H	V	C	H	V	C		H	V	C	H	V	C
0	5Y	8.3	0.3	5Y	7.3	0.7	0	5Y	8.3	0.3	5Y	7.3	0.7
0.5	5GY	7.8	0.6	5GY	7.0	0.6	0.5	5PB	7.7	0.3	N	7.0	0.0
1.0	5GY	7.3	1.0	2GY	7.0	0.8	1.0	5PB	7.3	0.6	N	7.0	0.0
2.5	8GY	6.8	2.2	5GY	6.4	1.5	2.5	7PB	6.8	2.6	5PB	5.5	1.1
5.0	8GY	6.5	3.0	7GY	6.0	2.7	5.0	6PB	5.4	4.0	5PB	5.3	2.7
10.0	10GY	5.9	3.5	8GY	5.4	3.0	10.0	6PB	4.6	6.0	5PB	4.7	4.1
20.0	9GY	5.1	4.0	9GY	4.8	3.6	20.0	6PB	4.0	7.2	6PB	4.1	6.0
35.0	9GY	4.6	4.0	9GY	4.4	3.8	35.0	6PB	3.3	7.7	6PB	3.5	6.7
50.0	10GY	4.4	4.2	10GY	4.1	4.0	50.0	6PB	3.2	8.3	6PB	3.1	7.5

	Iron Oxide Black (8242)							Carbon Black (8241)					
	White Cement			Gray Cement				White Cement			Gray Cement		
	H	V	C	H	V	C		H	V	C	H	V	C
0	5Y	8.3	0.3	5Y	7.3	0.7	0	5Y	8.3	0.3	5Y	7.3	0.7
0.5	N	6.9	0.0	N	7.0	0.0	0.1	N	6.7	0.0	N	6.1	0.0
1.0	N	6.3	0.0	N	6.2	0.0	0.2	N	6.0	0.0	N	5.8	0.0
2.5	N	4.4	0.0	N	5.1	0.0	0.5	N	4.6	0.0	N	5.0	0.0
5.0	N	4.1	0.0	N	3.9	0.0	1.0	N	3.9	0.0	N	3.9	0.0
10.0	N	3.5	0.0	N	3.1	0.0	2.0	N	3.3	0.0	5PB	3.5	0.5
20.0	N	2.5	0.0	N	2.7	0.0	4.0	N	2.8	0.0	3PB	3.1	0.7
35.0	N	2.2	0.0	N	2.2	0.0	7.0	N	2.6	0.0	5PB	2.7	0.7
50.0	N	2.1	0.0	N	2.0	0.0	10.0	N	2.5	0.0	5PB	2.6	0.7

cements, except where the quantity was so low as to change the color only slightly from that of similar mortar without pigment. This uniformity of the hue permits plotting the chroma and value of the colors produced by increasing quantities of pigment on one vertical plane of hue, with values

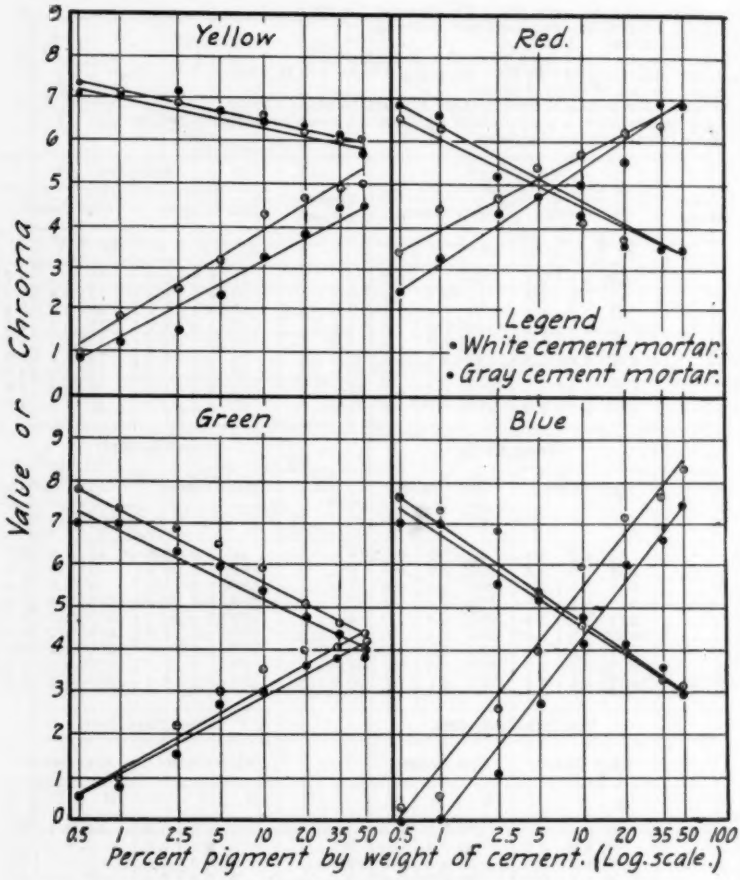
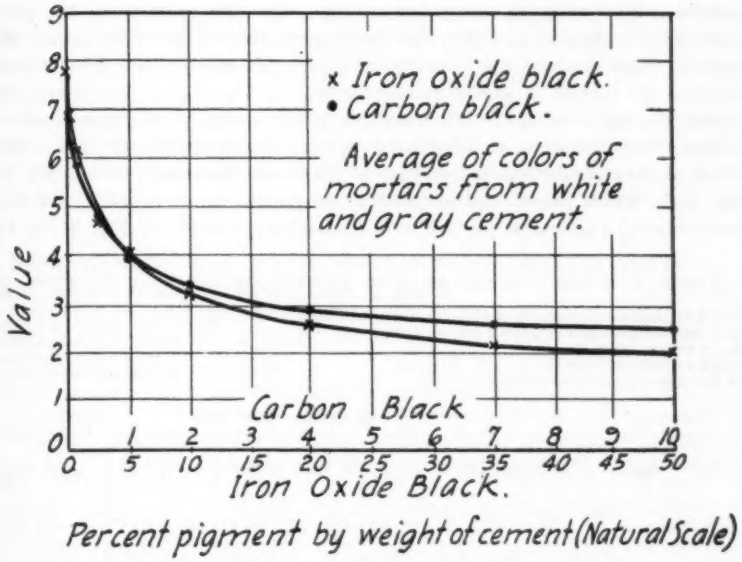

Yellow
Red.
Green
Blue
Legend
• White cement mortar.
• Gray cement mortar.
Value or Chroma
0 1 2 3 4 5 6 7 8 9
0.5 1 2.5 5 10 20 35 50 0.5 1 2.5 5 10 20 35 50 100
Percent pigment by weight of cement. (Log. scale.)

FIG. 3.—EFFECT OF QUANTITY OF PIGMENT ON COLOR OF 1:3 MORTAR.
Data from Table IV. Compare Fig. 2.

as ordinates and chromas as abscissas. (Fig. 2). The curves in the various parts of Fig. 2 are thus on different vertical planes of the Munsell color solid, with the value axis common to all and representing the axis of the system or scale of neutrals. The color of both the white and gray cement mortar without pigment was of yellow or yellow-red hue of weak chroma. In plotting the effect of the blue pigment on the color of the


x Iron oxide black.
• Carbon black.
Average of colors of mortars from white and gray cement.
Value
Carbon Black
Iron Oxide Black.
Percent pigment by weight of cement (Natural Scale)

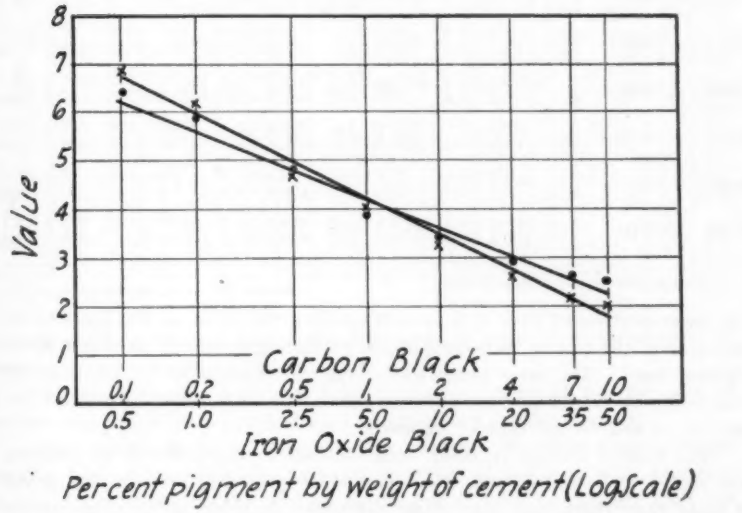


FIG. 4.—EFFECT OF QUANTITY OF BLACK PIGMENT ON COLOR OF 1 : 3 MORTAR.
Data from Table IV.

mortar the chroma of the uncolored mortar is placed to the left of the neutral axis or is given a negative chroma, since the purple-blue hue produced by the pigment is nearly the complement of the hue of the uncolored mortar. The effect of the quantity of these pigments on the chroma and value of the mortar is shown in another way in Fig. 3. In this figure the curves sloping downward with increases in the quantity of pigment show changes in value while those sloping upward represent the chroma. The effect of iron oxide black and carbon black on the color are shown in Fig. 4, in which percentage of pigment is plotted against value. If the percentage of pigment is plotted on a logarithmic scale as in Fig. 3 and in

TABLE V—EFFECT OF QUANTITY OF CEMENT ON COLOR OF MORTAR.

Color determinations on mortar discs 4½ in. diameter, 1 in. thick.
Age 7 days, 5 days moist curing and 2 days air drying.
Mix by volume; white cement.
For other details see Table IV.
For description of pigments see Table II.

Pigment		Color of Specimen—Age 7 Days															
Hue	Sample	Per Cent	Neat (0.40)*			1:2 (0.67)*			1:3 (0.88)*			1:4 (1.15)*			1:5 (1.42)*		
			H	V	C	H	V	C	H	V	C	H	V	C	H	V	C
		0	N	8.7	0.0	N	8.7	0.0	5Y	8.4	0.3	5Y	8.2	0.5	5Y	8.1	0.7
Yellow	8252	5	9YR	7.1	3.9	7YR	7.0	3.6	9YR	6.9	3.9	10YR	7.2	3.3	10YR	7.2	3.1
		10	9YR	6.6	4.5	9YR	6.7	4.4	10YR	6.8	3.9	9YR	6.9	4.1	9YR	7.0	4.1
Red	8240	3	7R	5.1	5.5	7R	5.3	5.1	7R	5.2	5.0	6R	5.5	4.9	8R	5.7	4.7
		6	8R	4.4	5.5	8R	4.5	5.4	7R	4.5	5.5	7R	4.8	5.1	7R	4.9	4.9
Green	8253	3	10GY	6.4	3.3	10GY	6.6	2.9	10GY	6.6	2.6	10GY	7.1	2.2	10GY	7.1	2.2
		6	1G	5.8	3.5	1G	6.1	3.3	9GY	5.9	3.1	10GY	6.3	2.8	1G	6.3	2.8
Blue	8619	3	6PB	5.8	3.9	6PB	6.2	3.4	5PB	6.2	3.1	6PB	6.7	2.4	6PB	6.9	2.3
		6	7PB	4.9	6.0	6PB	5.3	5.3	6PB	5.3	5.0	6PB	6.2	4.0	6PB	6.1	3.8
Black	8242	3	N	3.5	0.0	N	3.7	0.0	N	4.3	0.0	N	4.6	0.0	5R	4.6	0.1
		6	5R	3.0	0.1	5R	3.1	0.1	5PB	3.5	0.2	N	3.5	0.0	5R	3.7	0.1
Black	8241	1	5PB	3.6	0.3	5PB	3.5	0.2	5PB	3.7	0.2	5PB	4.2	0.2	5PB	4.2	0.2
		2	5PB	3.4	0.4	5PB	3.1	0.3	5PB	3.0	0.3	5PB	3.8	0.3	5PB	3.6	0 3

* Average water-cement ratio by volume.

the lower portion of Fig. 4, it is evident that the effect of the pigment on the color of the mortar is in proportion to the logarithm of the quantity of pigment used. The same numerical change in chroma or value would be observed between 1.0 and 1.1 per cent admixture as between 10 and 11 per cent or 50 and 55 per cent of pigment.

The rate of change in color with increase in the quantity of pigment was such for the yellow, red, green and black pigments used in this group of tests that there was very little difference in color between specimens made with 35 per cent pigment and those with 50 per cent pigment. The ultramarine blue had apparently not reached its limit at 50 per cent admixture.

In the case of the two blacks, these data show that one unit by weight of the carbon black is approximately equal in effect on color to five units of the iron oxide black. The iron black was one of the best of its kind used in these tests, while the carbon black was about the average of its kind.

For large quantities of pigment the color of gray cement mortar differed less than might have been expected from that of white cement mortar; for the smaller amounts of pigment, the differences were more pronounced. In some cases, it appears that practically the same color of

TABLE VI—EFFECT OF QUANTITY OF WATER ON COLOR OF MORTAR.

Color determinations on mortar discs 4½ in. diameter 1 in. thick.
Age 7 days, 5 days moist curing and 2 days air drying.
Mix—1:3 by volume; white cement.
Consistency measured by flow on 18-in. flow table.
For other details see Table IV.
For description of pigments see Table II.

Pigment			Color of Specimen—Age 7 Days											
Hue	Sample	Per Cent	Flow—105 (0.71)*			Flow—120 (0.79)*			Flow—160 (0.88)*			Flow—215 (1.00)*		
			H	V	C	H	V	C	H	V	C	H	V	C
		0	5Y	8.1	0.1	5Y	8.3	0.4	5Y	8.4	0.3	N	8.5	0.0
Yellow...	8252	5	10YR	6.6	4.0	9YR	6.7	4.0	9YR	6.9	3.9	10YR	7.1	3.8
		10	10YR	6.7	4.1	10YR	6.8	4.1	10YR	6.8	3.9	10YR	6.9	3.5
Red.....	8240	3	6R	5.2	5.3	6R	5.2	5.2	6R	5.2	5.0	6R	5.4	4.7
		6	7R	4.2	5.8	7R	4.4	5.5	7R	4.5	5.5	6R	4.6	5.4
Green....	8253	3	8GY	6.4	2.6	9GY	6.4	2.6	10GY	6.6	2.6	10GY	7.1	2.6
		6	9GY	5.8	3.2	9GY	5.9	3.1	9GY	5.9	3.1	9GY	6.3	3.0
Blue.....	8619	3	5PB	6.1	3.5	5PB	6.1	3.2	5PB	6.2	3.1	6PB	6.4	3.0
		6	5PB	5.1	5.3	6PB	5.2	5.1	6PB	5.3	5.0	6PB	5.4	4.9
Black....	8242	3	N	3.7	0.0	N	3.9	0.0	N	4.3	0.0	N	4.5	0.0
		6	N	3.1	0.0	N	3.4	0.0	5PB	3.5	0.2	5PB	3.7	0.4
Black....	8241	1	5PB	3.5	0.3	5PB	3.7	0.3	N	3.7	0.0	5PB	4.0	0.3
		2	5PB	3.0	0.3	5PB	2.9	0.4	5PB	3.0	0.3	N	3.3	0.0

* Average water-cement ratio by volume.

mortar can be produced with light gray cement as with white cement by using a larger quantity of pigment.

Specimens in this group of tests have been exposed too recently to furnish any information regarding their behavior on weathering.

Effect of Quantity of Cement.—The mortars of mixes varying from neat to 1:5 and containing the same quantity of pigment with reference to the cement were nearly enough of the same color to indicate that the quantity of pigment in mortar should be given in terms of the cement rather than of any of the other constituents of the mortar (Table V). In general, the leaner mixes have colors higher in value and weaker in

chroma; the extreme differences for all pigments except the blue are less than a unit step of chroma or value. The blue exhibits greater differences in both properties, particularly in chroma. It is probable that some of the pigment is utilized in coloring the finer portion of the aggregate. The blue pigment being of a hue complementary to that of the aggregate is affected to a greater extent than the other pigments. The use of an aggregate containing more fine material would doubtless result in greater differences in chroma between mortars of different mixes.

Exposure tests of this group have not been under way long enough to warrant any discussion of the effect of mix on color permanence.

Effect of Consistency of Mortar.—The water content of colored 1:3 mortar affects the appearance in much the same way that it affects that

TABLE VII—EFFECT OF QUANTITY OF PIGMENT ON COMPRESSIVE STRENGTH OF MORTAR.

Compression tests of 2 x 4-in. cylinders.
Strength ratios (per cent of strength of mortar without pigment).
Age at test; 28 days.
Compressive strength for white cement 2,620 lb. per sq. in.; gray cement 3,100 lb. per sq. in.
Mix—1:3 by volume.
Consistency to give flow of about 160 on 18-in. flow table.
Portland cement; white or gray, the latter a mixture of 4 brands purchased in Chicago.
Aggregate: calcareous sand from Elgin, Ill., graded 0 to No. 4 (F. M. — 3.00).
Pigment: per cent by weight of cement.
Cylinders stored under water until test; tested damp.
Each value is the average of 4 tests made on 2 different days.
For description of pigments see Table II.

Pigment, per cent	Yellow (8252)		Red (8240)		Green (8253)		Blue (8619)		Black (8242		Pigment, per cent	Black (8241)	
	White	Gray	White	Gray	White	Gray	White	Gray	White	Gray		White	Gray
0.....	100	100	100	100	100	100	100	100	100	100	0.....	100	100
10.....	115	111	114	118	112	107	124	153	115	107	1.....	99	91
20.....	102	102	115	115	100	109	140	156	112	114	2.....	88	80
35.....	93	94	93	99	100	84	160	127	106	111	4.....	66	69
50.....	71	81	74	92	88	80	145	169	100	97	7.....	48	50
											10.....	34	39

of uncolored mortar. The hue and chroma are practically the same for all consistencies from very dry to a rather sloppy mix. Value is the only characteristic which appears to be affected in any regular manner, the wet mortars being somewhat lighter with an average difference between very wet and very dry mortars of about one-half unit of value (Table VI). The difference in color of stucco finish coats due to variations in "suction" of the base coats is doubtless a difference in value due to differences in water content of the finish coat mortar caused by unequal absorption. Within the limitations of the method of color matching, it seems that the variations in *value* between different mixes of the same consistency or different consistencies of 1:3 mortar are dependent upon the water-cement ratio.

No information regarding the effect of water content on color permanence is available at this time.

TABLE VIII—EFFECT OF PIGMENTS AND AGE AT TEST ON MORTAR STRENGTH.

Compression tests of 2 x 4-in. cylinders.
Strength ratios of mortar containing pigments (per cent of strength of mortar without admixture).
Water-cement ratio: 0.79 for white cement mortar, 0.84 for gray cement mortar.
Cylinders on hand for test at 1, 2 and 5 years.
See Table VII for further details.
Each value is the average of 3 tests made on different days.

Pigment			White Cement Mortar			Gray Cement Mortar		
Hue	Sample	Per Cent	7 days	28 days	3 mos.	7 days	28 days	3 mos.
		0	100 (1900)*	100 (3420)*	100 (4580)*	100 (2070)*	100 (3610)*	100 (4860)*
Buff	8297	5	102	97	99	112	119	113
		10	107	96	98	128	125	111
Yellow	8252	5	110	95	99	124	121	123
		10	119	104	101	126	120	112
	8245	5	105	96	93	113	107	114
		10	119	103	103	117	107	110
Red	8240	3	107	96	104	109	110	100
		6	123	89	97	112	113	111
	8251	3	123	106	100	106	96	101
		6	72	103	112	87	99	105
	8257	3	119	113	108	106	100	105
		6	109	110	111	96	96	108
	8237	3	79	72	74	94	85	88
		6	88	73	73	92	86	81
Green	8253	3	98	89	98	101	98	97
		6	94	100	95	104	103	100
	8238	3	110	94	85	104	114	98
		6	116	90	89	129	121	103
Blue	8239	3	107	84	92	115	119	108
		6	124	96	88	138	126	95
Brown	8259	3	93	92	96	119	113	99
		6	102	93	88	106	111	108
	8254	3	105	94	92	117	107	101
		6	114	100	98	105	109	103
Black	8258	3	95	92	96	102	112	100
		6	88	92	93	111	122	105
	8242	3	104	80	92	114	92	103
		6	105	92	94	103	106	114
	8303	3	90	100	98	99	100	110
		6	100	96	100	104	109	105
	8298	1	107	90	98	100	109	101
		2	102	87	90	97	109	105
	8241	1	92	80	78	93	96	96
		2	81	79	82	93	97	99

* Compressive strength, lb. per sq. in.

Effect of Curing Condition and Admixtures of Hydrated Lime.—The differences in the color of the mortar from the same batch which was cured under different conditions were so slight that no attempt is made to present them in detail. No marked differences in the condition of the similar mortar variously cured could be observed after 6 mo. exposure to weather.

Admixtures of 10 per cent and 20 per cent of hydrated lime in addition to the pigments did not, in general, cause differences in color which could be detected, nor were there any observable differences in color permanence during 3 mo. exposure of gray cement mortar or 6 mo. exposure of white cement mortar.

TABLE IX—EFFECT OF QUANTITY OF CEMENT ON STRENGTH OF MORTARS.

Compression tests of 2 x 4-in. cylinders.
Strength ratios of mortar (per cent of strengths of similar mortar without pigment).
Age at test 28 days.
Mix by volume.
White cement.
Water-cement ratio by volume.
For other details see Table VII.
Each value is the average of 4 tests made on 2 different days.

Pigment			Water-Cement Ratio					Strength Ratio, per cent				
Hue	Sample	Per cent	Neat	1:2	1:3	1:4	1:5	Neat	1:2	1:3	1:4	1:5
		0	0.37	0.66	0.88	1.16	1.43	100 (10280)*	100 (4760)*	100 (2880)*	100 (1760)*	100 (1080)*
Yellow.	8252	5	0.39	0.67	0.88	1.16	1.42	114	112	106	107	106
		10	0.42	0.68	0.88	1.14	1.41	102	103	102	93	97
Red....	8240	3	0.39	0.67	0.88	1.19	1.44	102	102	110	95	91
		6	0.41	0.68	0.88	1.19	1.44	93	100	108	89	81
Green..	8253	3	0.37	0.65	0.88	1.19	1.44	121	106	111	97	101
		6	0.38	0.65	0.88	1.10	1.38	109	98	103	103	110
Blue...	8619	3	0.36	0.65	0.88	1.10	1.38	113	109	105	102	117
		6	0.38	0.65	0.88	1.10	1.38	106	116	116	122	129
Black..	8242	3	0.38	0.65	0.88	1.10	1.38	112	102	108	83	89
		6	0.39	0.65	0.88	1.10	1.38	110	100	113	87	99
Black..	8241	1	0.44	0.70	0.88	1.20	1.46	92	96	78	84	80
		2	0.48	0.75	0.88	1.20	1.46	69	84	79	82	75

* Compressive strength, lb. per sq. in.

DISCUSSION OF STRENGTH DATA.

Effect of Quantity of Pigment.—The effect of large quantities of pigment on the compressive strength of mortar is shown in Table VII. Due to an error in this group which was not discovered until too late to repeat the tests, the mortars containing no pigment did not have the desired flow. However, sufficient data were available on the mixing water-flow-strength relations of this particular mixture to enable a correction to be made on the compressive strength. The corrected compressive strength was used in calculating strength-ratios. In the cases of the yellow, red and green pigments, admixtures in excess of 20 per cent were required to produce reductions in strength, while 50 per cent admixture of iron oxide black

gave strengths equal to or only slightly lower than those obtained without admixture. Ultramarine blue has been recognized as having some puzzolanic properties, which account for the increased strengths due to its use.

It may be that these pigments absorb enough water to cause a lowering of the true water-ratio resulting in increased strengths for the smaller quantities of admixtures while with larger quantities the increased water necessary to obtain the desired workability and the diluent effect of the pigment became effective. The yellow and red pigments required greater quantities of water to give mortar of the desired flow than the green, blue

TABLE X—EFFECT OF CONSISTENCY ON MORTAR STRENGTH.

Compression tests of 2 x 4-in. cylinders.
Strength ratios of mortar (per cent of strengths of similar mortar without pigments).
Age at test: 28 days.
Mix—1:3 by volume.
Consistency measured on 18-in. flow table.
White cement.
Each value is the average of 4 tests made on 2 different days.

Pigment			Strength Ratio, per cent			
Hue	Sample	Per Cent	Flow 105 (0.71)*	Flow 120 (0.79)*	Flow 160 (0.88)*	Flow 215 (1.00)*
		0	100 (2430)†	100 (3220)†	100 (3040)†	100 (2120)†
Yellow	8252	5	102	110	106	111
		10	114	95	102	114
Red	8240	3	106	107	110	111
		6	97	112	108	112
Green	8253	3	126	119	111	140
		6	118	109	103	117
Blue	8619	3	123	118	105	110
		6	132	120	116	141
Black	8242	3	107	106	108	117
		6	114	119	113	118
Black	8241	1	81	94	78	95
		2	60	73	79	83

* Water-cement ratio by volume.
† Compressive strength, lb. per sq. in.

and iron oxide black, and with 35 per cent and 50 per cent admixtures became decidedly "sticky."

No tests of the effect of grading of the aggregate were included in this series. From general considerations it would seem that the aggregate used may have been somewhat deficient in fines, a deficiency which the pigment served to correct, making the mix somewhat more plastic. These speculations are presented to warn the reader against too broad an application of these strength results. It appears that under some conditions the use of quantities of pigment in excess of 10 per cent, sometimes recommended as the upper limit, is justifiable from the standpoint of its effect on strength and that under these conditions the real limiting factor is not that of

strength but of economy since the general run of pigments will probably give little increase in the intensity of color if more than 20 per cent is used. The use of large quantities of pigments high in calcium sulphate is probably not safe.

The case of carbon black appears to be different from that of the mineral colors. Under conditions presumably favorable to the production

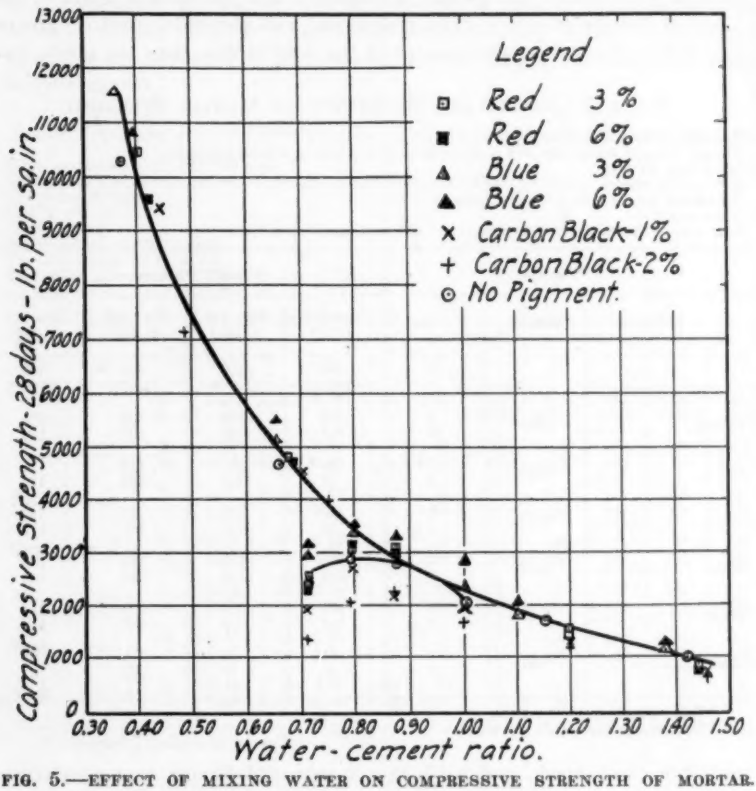


FIG. 5.—EFFECT OF MIXING WATER ON COMPRESSIVE STRENGTH OF MORTAR.

Data from Tables IX and X.

Plotted points for water-cement ratios 0.71, 0.79, 0.88 and 1.00 are for 1:3 mortars having flows of 105, 120, 160 and 215. Other groups of points along the upper curve are for the following mixtures with flows of 160; neat cement, water-ratios 0.36 to 0.48; 1:2 mortar, 0.66 to 0.75; 1:4 mortar, 1.10 to 1.20; 1:5 mortar, 1.38 to 1.46.

of high strengths with admixtures, 2 per cent of carbon black reduced the strength about 15 per cent, and 4 per cent reduced the strength about 30 per cent, while increases in strength were obtained with 10 per cent and 20 per cent iron oxide black, approximately equal in effect on color to 2 per cent and 4 per cent of the carbon black. This would indicate that, in general, iron oxide black is preferable to carbon black as a pigment for cement.

TABLE XI—EFFECT OF PIGMENT AND CURING CONDITION ON COMPRESSIVE STRENGTH OF MORTAR.

Compression tests of 2 x 4-in. cylinders.
Age at test: 28 days.
Mix—1:3 by volume, white cement.
Consistency to give flow of 160 on 18-in. flow table.
Water-cement ratio by volume: 0.79.
Cylinders cured under water, or in air of laboratory after 1 day in forms.
Strength ratios are percentages of strength of mortar without pigments.
For further details see Table VII.
Each value is the average of 3 or 6 tests.

Pigment			Compressive Strength, lb. per sq. in.		Strength Ratio, per cent		Ratio of Strength of Air-Cured to Strength of Water-Cured Mortar, per cent
Hue	Sample	Per Cent	Water Cured	Air Cured	Water Cured	Air Cured	
		0	3420	2430	100	100	71
Buff	8297	5	3320	2710	97	111	82
		10	3290	2780	94	114	84
Yellow	8252	5	3260	2480	95	102	76
		10	3570	2920	104	120	82
	8245	5	3270	2570	96	106	79
		10	3530	2730	103	112	77
Red	8240	3	3270	2410	96	99	74
		6	3040	2600	89	107	86
	8251	3	3610	2540	106	104	70
		6	3530	1630	103	67	46
	8257	3	3890	2460	113	101	63
		6	3770	2180	110	90	58
	8237	3	2460	2070	72	85	84
		6	2490	1900	73	78	76
Green	8253	3	3050	2600	89	107	85
		6	3430	2580	100	106	75
	8238	3	3210	2580	94	106	80
		6	3070	2810	90	116	92
Blue	8239	3	2890	3170	84	130	110
		6	3210	3130	96	129	97
Brown	8259	3	3150	2700	92	111	86
		6	3170	2590	93	107	82
	8254	3	3230	2860	94	108	88
		6	3420	2560	100	105	75
Black	8258	3	3240	2260	92	93	70
		6	3230	2360	92	97	73
	8242	3	2720	2390	80	98	88
		6	3140	2430	92	100	78
	8303	3	3410	2260	100	93	66
		6	3290	2440	96	100	74
	8298	1	3060	2430	90	100	79
		2	2970	2310	87	95	78
	8241	1	2750	2090	80	86	76
		2	2690	2080	79	86	77
							Av. 78

TABLE XII—EFFECT OF PIGMENTS ON COMPRESSIVE STRENGTH OF MORTAR CONTAINING HYDRATED LIME.

Compression tests of 2 x 4-in. cylinders.
Age at test: 28 days; cylinders stored under water until test; tested damp.
Mix—1:3 by volume; white portland cement.
Hydrated lime: per cent by volume of cement.
See Table VII for other details.
Each value is the average of 3 or 4 tests.

Pigment			Strength Ratio, per cent			
Hue	Sample	Per Cent	Lime, 0 per cent	Lime, 10 per cent	Lime, 20 per cent	Average
		0	100 (3420)*	100 (3010)*	100 (3040)*	100
Buff	8297	5	97	109	102	103
		10	96	105	105	102
Yellow	8252	5	95	103	106	101
		10	104	110	104	106
	8245	5	96	107	97	100
		10	103	110	110	108
Red	8240	3	96	109	104	103
		6	89	106	105	100
	8251	3	106	119	112	112
		6	103	121	108	111
	8257	3	113	114	103	110
		6	110	116	98	108
	8237	3	72	84	75	77
		6	73	78	74	75
Green	8253	3	89	105	94	96
		6	100	107	96	101
	8238	3	94	100	96	97
		6	90	102	111	101
Blue	8239	3	84	98	107	96
		6	96	98	105	100
Brown	8259	3	92	102	105	100
		6	93	95	97	95
	8254	3	94	102	101	99
		6	100	106	104	103
Black	8258	3	92	100	109	100
		6	92	103	102	99
	8242	3	80	97	98	92
		6	92	103	98	98
	8303	3	100	96	107	101
		6	96	103	111	103
	8298	1	90	98	97	95
		2	87	89	92	89
	8241	1	80	89	78	82
		2	79	87	76	81

* Compressive strength, lb. per sq. in.

Effect of Age at Test.—The effect of 17 representative pigments on the compressive strength of mortar at 7 and 28 days and 3 mo. is shown in Table VIII. Strength ratios only are given for the mortars with admixtures of pigments. The same quantities of the pigments of different hues were added as were used in the bulk of the exposure tests. These small quantities of pigments in most cases did not cause large changes in strength.

One red pigment (sample 8251) which contained calcium sulphate gave increased 7-day strength with 3 per cent admixture and decreased 7-day strength with 6 per cent admixture. Strengths at other ages were not far from normal. Other tests* show that gypsum has a similar effect as an admixture in concrete.

The ultramarine blue (not the same sample used in the tests on effect of quantity of pigment) caused increases in the strength at earlier ages and smaller increases or decreases at later ages. A green pigment (sample 8238), which is a mixture of ultramarine blue and a yellow pigment, showed similar effects on the strength at various ages.

Effect of Quantity of Cement.—The effect of pigments on the compressive strength of mortar of various mixes is shown in Table IX. Data for the red, blue and carbon black pigments are shown in Fig. 5, in which the compressive strength is plotted against water-cement ratio.

It will be seen that, in most cases, the pigments used in the quantities added had comparatively little effect on the strength, except as they affected the quantity of water necessary to obtain the desired workability. Ultramarine blue gave strength results among the higher ones for each mix, while carbon black gave strengths generally lower than that of uncolored mortar. The strength reductions caused by carbon black appear to be due partly to a specific action of the pigment, and partly to the necessity of using increased quantities of water to obtain workability.

Effect of Consistency.—The compressive strengths of the very dry mortars were well below the water-ratio curve obtained from the strengths of the mortars of different mixes, and those of the mortars with a flow of 120 fell slightly below it. (Table X and lower curve of Fig. 5). The first mix was extremely harsh, and the other appeared to be somewhat less workable than was consistent with easy placing. The mortars of the very wet consistency fell on the water-ratio curve, the blue pigment giving somewhat high and the carbon black somewhat low strengths.

Effect of Curing Condition.—Table XI shows the effect of curing in water and in air on the compressive strength of colored mortar. Both sets of cylinders were kept in the forms about 18 hours; one set was stored under water until test and tested damp; the other was kept in the air and tested dry. The average 28-day compressive strength of all the air-cured cylinders was 78 per cent of that of those cured in water. Most of the

* Table 15, Bulletin 8, Structural Materials Research Laboratory, Lewis Institute, Chicago, "Effect of Hydrated Lime and Other Powdered Admixtures in Concretes," by Duff A. Abrams.

individual ratios were close to this figure. The two red colors which contained large quantities of calcium sulphate gave particularly low strengths in the air-cured mortar, especially the 6 per cent admixture of Lot 8251, the air-cured mortar having 67 per cent of the strength of air-cured mortar without color and 46 per cent of the strength of similar mortar moist-cured. Air-cured mortars containing ultramarine blue had relatively high strengths as compared to average of air-cured mortars; in one case, that of the 3 per cent of pigment, the strength of the air-cured mortar was actually higher than that of the moist-cured mortar. The composition of the pigment seems to furnish a tentative explanation of this behavior. Ultramarine blue contains silica combined in a manner which would be expected to give it some cementing properties akin to those of puzzolanic materials. The sodium sulphide which is present in ultramarine blue might be expected to have a weakening effect on portland cement under some conditions. It seems quite possible that continued wet curing augments the weakening effect of the sodium sulphide while it has a lesser effect on the development of the puzzolanic properties of the silicate. This hypothesis would account for the relatively greater increases in strength for air-cured than for water-cured mortar.

Effect of Hydrated Lime.—The effect of the pigments on the 28-day compressive strength was generally about the same whether the mortar contained 0, 10 or 20 per cent hydrated lime. (Table XII). The average of the compressive strengths of all mortars containing lime was slightly lower than that of the mortars without lime, being 96 per cent for the 10 per cent admixture and 95 per cent for the 20 per cent admixture. The groups containing various quantities of lime were made at different times and the average water-cement ratio for the mortar containing lime was enough higher than that of the mortar without hydrate to account for the difference in strength. The minor differences in grading of sand from day to day were sufficient to require changes in water to insure equal workability. These tests can hardly be considered as having any bearing on the question of change of workability due to the addition of hydrated lime.

Tests of Colors for Mortars—Discussion.

Mr. Earley. JOHN J. EARLEY.—Those of you who make concrete products seriously as a living and who have attempted to mix pigments with cement in that process, have without doubt experienced something never to be forgotten. Mr. Wilson's paper is a definite and valuable contribution of much-needed knowledge on that subject.

Mr. Toch. MAXIMILIAN TOCH.—The use of pigments mixed with portland cement was something I discussed here two years ago, and it created considerable discussion. I came to similar conclusions, that calcium sulphate or gypsum added to any pigment weakens it considerably. In this paper we have heard that ultramarine blue increases the tensile strength of portland cement concrete inordinately. That is quite true, but I think that statement can be qualified because there are 57 varieties, more or less, of ultramarine blue. There are some ultramarine blues which are made with calcium sulphate, and those I found had a weakening effect rather than a beneficial effect. However, the average ultramarine blue which has been washed of soluble salts is much superior to that made in the ordinary way by means of gypsum. Ultramarine blue is a synthetic, semi-precious jewel —lapis lasuli. It is a pigment used as far back as 2000 B. C. There are any number of examples still existing in Egypt where the jewel was ground, mixed with white of egg or glue, used as a pigment, and it is as bright today as it was then. A prize was offered in France in 1829 for the synthetic manufacture of this pigment because the genuine was much too expensive for artists to use, and eventually it was made and the present ultramarine blue has every good qualification of the original semi precious jewel.

Its use in the concrete is remarkable. I had our laboratory take every one of the raw materials from which ultramarine blue is made, and make tensile strength and color tests, and we have thus far been unable to find that the material is in ultramarine blue that makes it such a valuable addition to portland cement concrete. I can say that if we knew and we found we could decolorize it, it would be one of the best waterproofing and strengthening materials for concrete that has ever been found.

I cannot quite agree with the fact that all of the organic colors are useless in portland cement. I do not wish to go into their real names, although I might tell you that one that I found, a very bright scarlet known as fire red, has exceptional qualities, but when the concrete is exposed for six months, shows a slight fading. You will always find that on rubbing the concrete with varnish the pristine color comes back, which, evidently, as far as I can find out, is simply a masking of the color, due either to the slight crystallization of lime, or the formation of laitance.

There are other organic colors which are also good, but this much I can tell you: that the entire color industry, as far as portland cement is concerned, is probably not 0.01 per cent of the amount of portland cement used. If you stop to think of it, nearly every portland cement structure that you have seen is in its natural gray state or in its white state, if white portland cement was used, and the amount of color used was very small.

Mr. Wilk. BENJAMIN WILK.—It so happens that this last year I have had a great deal to do with colors of concrete in trying to market a common face brick in colors. The first question I would like to submit here is: Is there such a thing as an absolutely non-fading color that can be added to concrete? The question of colors is an exceedingly important one, and I think there is a great opportunity in the use of color with concrete on face brick alone. There is a tremendous market if we can produce a brick that has a color that is not fading, because the other requirements can be met: strength, absorption, size, price. The question of color, therefore, is the most important. Certainly you have to use the best color you can buy.

I am interested in what we can do with black. We tried carbon black, but that did not look as good as we thought it should. The addition of a waterproof paste seems to help a great deal. I am wondering whether anyone has had any experience with salts, aluminum sulphite, for example, in holding the color.

Mr. Lindstrom. ROBERT SETH LINDSTROM.—I have been greatly interested in the matter of colored concrete. I received some information today that I have been seeking for several years, that is, how to get the natural color of the concrete. I find by using muriatic acid and water when cleaning portland cement concrete the various aggregates together with the natural color of the stone, will become exposed, which is similar to the process when you clean Bedford stone with acid and water to bring out the original color of the stone, and I think that method of producing color is one of the best I have ever seen. In my experience in manufacturing reground portland cement for the last eight years, I have spent a great deal of money with technical engineers in trying to obtain a pigment that would mix with cement and retain its color in the finished product, and I have yet to find it.

Mr. Harrison. MERRITT HARRISON.—I have endeavored to work a little bit with color, and I find it is very discouraging to put up a building and go back and look at it two years afterwards and find it is all covered with soot and dirt. We were here at this convention last year, and this demonstration as to colored stucco was used in our home town and we produced some very creditable results there. We even showed the plasterer just how these colors were mixed on his palette, but at the end of about a year, with the soot and dirt that comes from the smoke, we have very discouraging results. I hope that there will be some way that we, as men interested in concrete, can urge the engineers interested in boiler construc-

tion to design so that smoke will be disposed of before it gets out in the air to spoil our color schemes.

LESLIE H. ALLEN.—The plant of which I am manager has been using a large amount of colors for several years. So far we have found the metallic oxides are the only ones suitable for use in coloring our roofing tile, but we also found that those which had a very high or a very low specific gravity do not give as good results. Carbon black is much lighter in weight than the cement, and I suppose the way it floats in the mixture causes it to float to the surface and wash off quite rapidly. **Mr. Allen.**

On the other hand, very heavy metallic colors, compounds of lead, etc., are no more satisfactory; you can rub them off with the finger after the tile has hardened. I do not know whether that is due to the so-called greasiness of the feel of the lead, or whether its weight causes it not to combine properly with the cement. Thoroughness of mixing is important, I believe, and I would suggest that tests on color mixed with cement for longer periods to find out what effect time has, would be very helpful.

The high-grade metallic oxides are ground at least twice as fine as the cement, and in order to secure thorough mixing, I believe a very long mixing time is necessary. We have found it satisfactory in our work. We do not grind our cement and color together; we use plenty of water and with a long time of mixing we can get a thorough mixing with very much less expense than if we put in grinding machinery.

Another important point is the purity of colors. The cheap reds seldom run more than 50 per cent ferric oxide. The Spanish oxides run up to 85 per cent, but the best precipitated oxides run up to 93 and 97 per cent, and give a much brighter color than the Spanish color.

I think these points, specific gravity, time of mixing, and purity of color, are very important problems that we encounter in trying to turn out a colored concrete product.

We feel pretty sure of the permanence of color of the commercially pure ferric oxide and chromium oxide. Browns have been referred to. There are two in use. In one of them, the roasted ore, containing 10 per cent or 15 per cent of manganese, we have observed a considerable change in shade in two or three years. Whether that is due to manganese separating, I do not know. The same applies to ultramarine blue. We buy a fairly good grade of ultramarine blue and over a period of a year we noticed a difference. Over three years there is some difference, but we feel sure of pure ferric oxide and pure green.

GEORGE T. WILHELM.—Was it the actual fading of the colors you observed, or efflorescence? **Mr. Wilhelm.**

MR. ALLEN.—Efflorescence is primarily a matter of curing. Providing you have a pure color to start with and that proper attention is given to curing, you get some efflorescence but not much, and that disappears in three to nine months. When I spoke of a permanent color, I meant the color of the tile as it leaves our plant, which is about the same color we **Mr. Allen.**

expect to find it twelve months later. There may be some efflorescence between those periods, but at the end of twelve months it is gone.

Dr. Toch. DR. TOCH.—It is evident that Mr. Allen knows what he is talking about, judging from his experience, because I have had similar experiences. The use of lead pigment in concrete should be prohibited. The action of it is fatal. You all know that portland cement when mixed with water and aggregate gets its ultimate strength through the liberation of lime, acting as a cementitious material, but the moment you add lead salts, the lead compound combines with the lime, and therefore robs the cement of its cementitious quality. You can rub off the surface when it is set, and it breaks just about with the tensile strength of two-day old rye bread.

As far as the purity of pigment is concerned, there is a great deal in that. Chromium oxide, which is recognized as the real green that should be used, is not so good unless it is almost 99 per cent. As it is made it contains a fairly large amount of borax and boric acid that is in the nature of its composition, and unless that is washed out, it is not quite such a good color. That accounts in a large measure for the failures and fading that one finds in chromium green. These borates crystallize and come to the surface and mask the color. As far as the reds and browns, which contain manganese, are concerned, that, too, has been explained, for the brown manganese is a lower oxide and as it gets old it becomes a higher oxide on account of its exposure to the air and the higher oxide is much deeper, almost black, as compared to the brown, which is not the same oxide as the one that is eventually produced. Iron oxides do exactly the same thing; certain iron oxides in the paint become very dark brown due to the oxidation and some suffer reduction.

It has been stated that manganese and black oxide of iron give better results than carbon; there isn't any doubt about that. The only trouble is that users of black will not pay the price for pure black oxide of iron, and black oxide of iron is 1/5 to 1/6 as strong as carbon black, yet a mixture of pure black oxide of iron and carbon black, from my experience, produces excellent results.

The Architectural Theory of Concrete Design.

Frederick L. Ackerman.*

At the outset it seems no more than fair to explain that I do not wish to pose as an expert in the field of concrete design. Experience in the design of structures where concrete has been employed in both a structural and decorative sense has been limited. Concrete comes into the view of the architect as one of an ever increasing list of building materials with which he has to deal. The very wealth of materials available tends to confuse rather than simplify his practice and clarify his point of view. Except in rare instances, the architect's knowledge of such a material as concrete is derived from a few isolated experiences such as occur in the course of his varied practice. So that the point of view from which my observations issue may be assumed to be that of the average architect. Naturally I am not attempting to speak for the profession: but I may voice what my contacts indicate constitutes a general opinion.

Concrete is not a new material in the world of building. Indeed, it is a material of great antiquity. But the conditions under which it is now employed, the technology of its use under the strict guidance of scientific insight and knowledge, shifts it over definitely into the category of something new. We may well pause for a moment and consider the significance of this change.

When a new material, device or process takes definite shape out of our experimental fumblings, there follows, particularly when the invention is of a revolutionary character, a long train of blundering effort in our attempt at adoption, use and development. The act of discovery and invention does not set us free: our habits and preconceptions bind us to our old techniques and applications. Recall how, in the transition from handicraft to machine technology, machines were first made to simulate the motions of the hands using the simple tools of handicraft. The conscious aim was the reproduction of manual operations on an enhanced scale, scope or pace. A century and a half has only partially freed us from the handicraft concept of production.

So long as the idea obtained that the new devices and processes were in the nature of substitutions, both the devices and the things produced retained the characteristics of what had gone before. The field of industry is filled with illustrations of how reluctant we have been to abandon the forms that had served, although under radically different circumstances.

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The first locomotives and first automobiles bore their heavy loads of features characteristic of the horse-drawn vehicles for which they served as substitutes. Many years elapse, following the first application of new mechanical principles, before we are able to clear away what later seem like the most obvious of hampering features carried over into the new technology by habits of thought that continued to run in terms of the process that had so given ground.

Without resorting to illustrations, we may say that much in the same way that the form of tools, the technique of use and the forms produced by handicraft industry were carried over into the era of machine technology, but to retard development, so the rules of thumb, the empirical formula of the handicraft period were carried over into the era of science but to hamper an objective handling of the facts under scientific inquiry.

All this may seem remote from the subject in hand: but it has a direct bearing on the case. For the use of this new material is now seriously hampered by the condition that the present objective with respect to its use centers in the utilization of it as a substitute. It is to be used as a substitute for wood, for brick, for stone or for steel, as the case may be. This holds true as regards structural and decorative use. This is not to suggest that concrete might not properly take the place of these several structural elements in so far as it is warranted by its properties. It is rather to note that its application is first conceived in terms of the fully developed, functional application of materials whose properties differ in a fundamental way. The designers of limousines and motor trucks are no longer hampered by preconceptions in the shape of victorias and lumber wagons. But the designers of lighting fixtures still think in terms of candles and oil lamps. Designers of concrete are as yet handicapped by what has gone before, much in the same way as are the designers of lighting fixtures. Structural engineers are not in the enviable position of the designers of aircraft: the science of aviation was not retarded in its development by the drag of habits and archaic techniques. Once the principles had been established there was a fair field ahead for development.

I have referred to concrete as a new building material. What serves to place it in a category by itself is the comparatively recent expansion of its structural functions. This expansion of functions was due to the application of science. I am not here referring to our modern methods of handling it: these are products of the machine process. I refer to the multitude of structural uses to which it is now put. No longer is its use confined to weight bearing. It now serves in tension as well as compression. Not only walls and piers but beams, girders, slabs, arches and vaults are all rational expressions of its properties.

This is, of course, but to state the obvious. But it may be worth while to emphasize it. For during the early phases of development, those engaged in experiments and those intimately associated with the use of an invention, a new material or process, are not likely to appreciate fully the significance of what they do as a matter of course.

The introduction of a single material that will serve a wide range of uses hitherto served by a variety of materials is significant from the standpoint of architectural expression. For it constitutes no less than the foundation of a revolutionary change in design.

We must not overlook the fact that the traditional divisions of architectural composition were derived from the properties of the materials used. And the traditional forms of architecture with which we are familiar grew out of the necessity of acknowledging such things as the limiting dimensions of materials available, convenient size, shape and weight to handle under handicraft technique, hardness, durability, limiting strength. These played a definite part in giving form and character to whatever was built.

The building of structures, stone by stone, brick by brick, gave rise to a pattern. Under the spirit of workmanship the utilization of this pattern and the development of it into a decorative expression was a matter of course. In the same way the development of certain horizontal lines and divisions were derived from the process of building. Corbels, brackets, the decorative features of cornices, architraves and the decorative treatment of doors and windows expressed both structural procedure and decorative aims. In the early expression of what we characterize as the Styles of Architecture it is gratuitous to attempt to fix the point at which aims passed from those centered in the matter of fact logic of construction to those involving decoration. The history of industry offers nothing which indicates more perfectly than the structures of the Gothic period how it is that out of an unhampered, workmanlike handling of materials, forms of a decorative character emerge, architecturally expressive of the materials dealt with.

But to return to the subject of concrete design. I am not now referring to that phase of design that concerns itself merely with loads and stresses but to a larger point of view which includes both the structural and decorative use of the material—that is to say, its architectural use.

For the purpose of more clearly exposing the problem of design let us assume that we have developed a surface of intrinsic value—a surface durable and pleasing. What would then constitute a rational use of this material? Instead of first proposing a theory, we may, to better advantage, suggest its underlying thought by proposing that we erect forms, such as would be used in the case of a building of concrete, about certain historical structures. We might try, for example, to erect concrete forms about the structures upon the Acropolis at Athens; we might also try to enclose one of the great Gothic cathedrals of Central Europe or we might attempt to build a concrete form about St. Peter's at Rome.

It is hardly necessary to dwell upon the difficulties involved in such a procedure; the very nature of the processes involved points to the absurdity of it and at the same time illuminates the first article of a theory of concrete design. Our theory of concrete design resolves itself immediately into a theory of building the preparatory structure, the form.

The preparatory structure—the form—is the beginning and the end of concrete design.

We are dealing with a plastic material. But as such its use as a building material differs in a fundamental way from that of clay when it is built up and worked into definite form by the hands of the modeller. In the erection of concrete structures we are not dealing with a process as simple and direct as modelling. But I suspect that the fundamental difference between the process of erecting concrete structures and modelling in clay has been overlooked in the same way that we have overlooked the fundamental difference as between concrete and stone. For in the case of many concrete structures, where the designers have sought consciously to depart from the whole range of architectural forms and masses that had their origin in stone or brick, it would seem that the underlying thought of the designers was the production of forms and masses that would indicate first of all the plastic quality of the material used. This is true in particular of the experimental work about the shores of the Baltic where many of the revolutionary designs suggest that they had been modelled in clay and baked in the sun like ancient pots.

In concrete we are dealing with a plastic material. But its use differs from that of clay in that it serves our structural and architectural ends only when it is rigidly confined within a preparatory structure—the form. So, I repeat for sake of emphasis that the construction of the form constitutes the beginning and the end of concrete design—the theory of concrete architecture centers in the theory of form building.

A new architectural expression of a revolutionary character cannot be generated at will. There must be, apparently, if we may draw upon the history of industry, a period of experimental work—a period of transition. And the outcome depends, not so much upon the potentialities of the case as it does upon the thought that guides exploration and creates definite aims. At most all that we can do is to work toward such aims as seem rational to us in the light of our experimental effort.

To state in broad outline the theory of form building we have: the form should be composed of simple planes such as would enclose the volumes required by our structural theory and as dictated by our general concept of what should constitute the mass as a whole. That is to say, our mass would be conceived in terms of simple planes. This gives us obviously something quite different from the architectural examples previously referred to.

Our simple masses may be modified in silhouette and in surface treatment in an endless variety of ways and for reasons that have little or no bearing upon the structural theory from which we derived our masses. Refinement and embellishment fall within the scope of concrete the same as other materials. We may properly introduce lines, secondary planes and ornamentation. But in the modification of our masses we must bear in mind that we are neither building in stone nor modelling in clay: we are building a form. And so whatever we do to achieve variety and interest

must commend itself as a perfectly rational procedure with respect to the building of the form itself.

Without going into the details of form building we may mention a few of the steps which would constitute a rational procedure. By secondary forms built into the angles of our preparatory structure we may produce champfers of endless variety and detail. Thus the silhouette of piers may be modified and shaped to our aims. In the same way the enrichment of openings—doors, windows, rectangular and arched—may be secured. These secondary forms in no wise complicate the construction of our primary form: nor do they weaken it. They strengthen the form and at the same time eliminate the sharp arrisses from the structure.

The simple planes of our primary form lend themselves to endless modification by the application of secondary forms which will appear in the finished structure as incised lines, sunken panels, plain or molded. Again these in no wise weaken or complicate the building of the primary form which remains undisturbed. Nor need we stop with the introduction of champfers, lines and panels. Still working within our form we may proceed to the treatment, in intaglio, of the secondary planes and even lines and champfers.

As a theory of form building, nothing could be simpler. But the difficulty arises in its application, for we are not accustomed to proceed to our end in this way. By a mental process more or less similar to that out of which the broad outlines of architectural masses have been established we may proceed to the design of the main outlines of our preparatory structure. But having established that, the process that follows must be that of the worker in intaglio. We may score the surfaces of our monolithic structure into imitation stone joints; we may indicate voussoirs over our openings to suggest stone arches, but when we do these and other similar things we merely expose our inability to think out the design in terms of the preparatory structure and this plastic material which takes it form by being confined.

The application of this theory of form building would give us an expression—a character similar in many respects to those which arose when the arts of building were young. For the use of simple planes, and ornamentation by incision, were characteristics of nearly all early architectural expressions. We may or we may not draw upon such sources for our inspiration. But it would seem sufficient if we were guided in our design by the thought that the process and material would be expressed in something like an ideal sense, were the greatest projection of our decorative features kept within the broad planes which serve to define the mass.

No reference has been made to the use of concrete cast in forms prior to placement in the building. So far as concerns the effect and hence, so far as concerns design, it can make but slight difference whether the material used be stone or cast concrete. Shapes and the dimensions of individual pieces might differ; and what has already been said of form building would apply with some force to the preparation of forms for individual

blocks of concrete. But even so, a theory of an architecture rendered in pre-cast concrete would differ in no material way from the theory of an architecture rendered in stone. For the technique of erection would be essentially the same in either case.

No reference has been made to a preferential category of structures which would lend themselves to a logical handling under this theory. Such an act would be entirely gratuitous; it would be to arbitrarily set an artificial limitation upon the application of the theory and it would be to prophesy an outcome which of a necessity must be the product of an evolutionary application. The types and kinds of structure that may be erected of concrete turn upon the development of a technique that will serve whatever utilitarian or æsthetic demands are ultimately laid upon the designers of buildings.

And finally: I have made no reference whatever to the matter of cost in discussing the architectural theory. It may be said that cost is a controlling factor. A rational design is one that achieves rational ends by direct and simple ways and means. A theory, therefore, that is based upon the use of simple, direct methods need not concern itself with other than the technology of the operation. Preoccupation with cost is likely to leave no room for exploration into the technological possibilities. This is well illustrated in the development of concrete.

The Use of Concrete on the Pacific Coast—Its Structural and Æsthetic Possibilities.

By John C. Austin.*

At the start, it may not be out of order to venture the statement that this subject is as big as it is important, so important, indeed, as to render it impossible to do it justice in a necessarily short discourse. It must, therefore, be left to the judgment of the convention to determine whether I have succeeded in adding any ideas of value to the general, as well as to the more particular information now so readily obtainable from many and varied reliable sources.

We all know that various kinds of concrete have been employed for centuries, and that many example may be found in different parts of the world today that conclusively prove the ancients well understood its use and its great value, but within certain limitations. Its growth and rapid development in this country may be said to date back about a generation or so and, during that period, the tremendous strides it has made, in nearly every direction, render it nothing short of phenomenal, warranting the prediction that, in course of time, it is destined to become a "universal" medium of structural expression.

The practicability of its use even in the construction of buildings of great height is splendidly demonstrated in the 17-story "Ingalls Building," erected at Cincinnati in 1903, and for many years the "tallest concrete office building" in the country. It was regarded, at the time, as a very daring undertaking, and there were many architects, as well as engineers, who were skeptical of its success. However, it is only reasonable to suppose that the architects must have convinced the owners that it was entirely safe to proceed with the operation, in spite of the fact that concrete construction, as applied to tall buildings, was never before attempted.

Naturally, at that time engineers, as well as architects, were inexperienced and more or less timid in the application of such rules and formulas as had been established up to that period and, for this reason, made their calculations on the side of safety.

If the same project was undertaken today, columns, girders, beams and slabs would, doubtless, assume very different proportions and produce a relatively larger rentable floor area and proportionate increase in income. This has been aptly demonstrated in the construction of office buildings during the past ten years, the most notable of which are the Arcade Building, St. Louis (17 stories), Hide and Leather Building, New York (18

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stories), and the Medical Arts Building, Dallas, Texas (20 stories). Many others might be mentioned.

But here, we are more concerned with the use of concrete on the Pacific Coast; and a tour of inspection from Seattle on the north to San Diego on the south will convince the most skeptical that reinforced concrete has come to be regarded as one of the best materials to be employed in the construction of safe and fire-resisting buildings.

In view of our remote distance from the steel manufacturing center and its consequent excessive cost, the more general and extensive use of structural steel in our building projects of considerable magnitude is almost prohibitive. Or, if not altogether prohibitive, at least these factors act as a deterrent when considered along with the question of reinforced concrete. Consequently, from an economical standpoint, reinforced concrete on this Coast stands out pre-eminently as a leader among structural materials.

Aside from this fact, however, it is also true that, while concrete is readily adaptable to many purposes, it cannot be employed in every case as efficiently or as economically as steel; and, even in the event it could be, it is not always advisable to give it the preference over steel construction when an intelligent combination of the two materials would best fit and solve the problem in hand.

Owing to our increased knowledge and improved technique of structural design, particularly with reference to fixed frames, arches and general problems dealing with continuity and monolithic conditions, together with the general increased efficiency and control of concrete production, the adaptability and use of reinforced concrete, as a strong and economical structural and architectural medium, is rapidly increasing and gradually tending towards the development of a new architectural style.

A great deal of the heretofore laborious mathematical work involved in its design is gradually being eliminated, while shorter and more practical methods of calculation are coming into vogue and, to a large extent, being standardized.

The simplification of the methods of design has tended towards the establishment of more confidence and a broader, clearer conception and knowledge of the properties of reinforced concrete which, in the last analysis, means that this type of structure will henceforth be more and more efficiently and economically designed.

The use of the principles of the arch, particularly in reinforced concrete, should be exceptionally well adapted to our structures. It is a fact that, generally speaking, no more than 37½ per cent of the concrete in a straight beam or girder is figured to resist compression, whereas close to 100 per cent of the concrete in an arch is usually effective in resisting compressive stresses. The arch is one of our most economical structural units and its use architecturally is legend.

From an engineering standpoint, the structural possibilities of reinforced concrete are practically without limit. Cantilevered footings along

property lines permit occupancy of the full width of a lot without encroaching on adjoining property.

General alterations and revisions in plans during the construction of reinforced concrete can be made more economically and expeditiously as compared to equivalent changes in steel construction wherein detailing and fabrication play so important a part.

Selection of the type of structural design may well be left to the judgment of the architects and engineers, who are to prepare the plans. Structural engineers are becoming more and more familiar with the best practice in reinforced-concrete design, and architects, in increasing numbers, are taking advantage of the characteristic qualities and adaptability of concrete, so that, today, we find many fine examples of concrete buildings architecturally attractive and structurally correct.

Close proximity to the natural sources of supply of cement, sand, rock and gravel, and the cheapness of reinforcing steel, compared to that of structural steel, explains why reinforced concrete will continue to be used on an ever-increasing scale on the Pacific Coast for all types of structures, regardless of their individual purposes.

That this is true with respect to the Los Angeles territory is clearly indicated by a tabulated report, giving the comparative number, cost or values of steel and concrete construction during the period of five years ending Dec. 31, 1926.

It shows that nearly five times as many structures were built of concrete as of steel, and that the aggregate cost of the concrete buildings was more than double that of the steel buildings. It also shows, and quite naturally so, that concrete was used in a greater variety of buildings solely by reason of its lower cost. In other words, as an economical expedient and frequently, perhaps, as a producer of proportionately increased revenue.

Year	Structural Steel		Reinforced Concrete		Per Cent of Total	
	Number	Cost	Number	Cost	Steel	Concrete
1922	5	$7,454,000	43	$7,014,940	52	48
1923	42	20,103,400	65	17,169,005	54	46
1924	15	5,550,300	74	18,646,287	22	78
1925	10	5,800,000	101	29,006,845	17	83
1926	6	3,776,000	72	18,832,670	17	83
Totals	78	$42,683,700	355	$90,669,747	..	..
Average	15.6	$8,536,740	71	$18,133,950	32	68

Now, as to the æsthetic possibilities of concrete it must be manifest, even to a casual observer, that the volume and variety of the uses of reinforced concrete have been steadily increasing and its ready adaptability to the production of architectural effects in buildings, aspiring to a more highly developed character, are just beginning to be appreciated.

Heretofore many architects have assumed that, if a building is really to aspire to any degree of importance architecturally, it must exhibit a veneer of a permanently superior nature, such as characterizes granite, stone, marble, terra cotta, etc. That this is a fallacy is effectively demonstrated by the fact that, in the rapidly growing city of Los Angeles, there are a number of progressive architects who are ardent advocates, even champions, of the more general employment of reinforced concrete for all types of buildings, including dwellings; and they have produced many unusual, remarkably beautiful and charming examples of a variety of types.

As a result of their leadership, the concrete fever has spread with amazing rapidity in this city and immediate vicinity; especially in connection with the planning of theaters, auditoriums, churches, etc., or, broadly speaking, buildings of extraordinary and unusual type, in which intricate structural problems were presented, the solution of which appear to have been reached by the simpler and more economical methods afforded by reinforced-concrete construction.

At this juncture, it might not be amiss to refer to a few of the most outstanding examples of the above classes. The Metropolitan Theater, designed by William Lee Woollett, shows, perhaps, the boldest and most original use of concrete for decorative purposes, and by some competent critics is declared to be the most successful of its kind in the country. I am not familiar enough with what has been done in this direction elsewhere to enable me to draw comparisons, but I am quite willing to agree that it is a very worthy and creditable undertaking.

Then, with due modesty, I take the liberty of directing attention to the Shrine Temple designed by me and erected in Los Angeles; a building that cost approximately $2,500,000, covering a ground area of 140,000 sq. ft. and possessing several noteworthy features. It consists of two units, an auditorium, together with its balcony and boxes, has a seating capacity of 6,700, and a banquet hall, with a balcony, will accommodate 5,000 persons at one sitting. This hall is 260 ft. x 140 ft. x 40 ft. high, while the auditorium is 192 ft. x 196 ft. x 63 ft. high. (At the moment "The Miracle" occupies the boards, and since the opening on Jan. 31, has tested the seating capacity at every presentation. This is true of opera and every other entertainment for which the building has been used during its first year).

Architecturally, its design would be classified as Saracenic. All the details were studied with the greatest care and were skilfully executed—while their beauty has been wonderfully enhanced by the judicious use of color incorporated in the concrete. Color treatment of concrete surfaces is a comparatively new art, which promises wonderful possibilities and may be divided and described as follows:

Color materials mixed with the aggregates; surface treatments, such as acid staining, applied oils or stains, and abrasive treatment, by tooling, to bring out the colors in the stone or gravel.

Stucco or plastered finishes, such as neat cement washes, application of patented stuccos, such as California colored plasters, scrafito and the cement gun. Inserts for monolithic casting, such as colored tiles, marbles, and other decorative materials.

To quote from an address entitled "The Revolutionary Effect of Concrete on Architecture," delivered by Professor Beresford Pite, F. R. I. B. A., at a meeting of the Royal Institute of British Architects recently, he said:

> The door is opened. The designer is no longer in a stonework prison. The realization of the possibility of the deliverance from ideal proportions established in masonry by the substitution of reinforced concrete is the taking of a very considerable step in the path of architectural progress. The dead hand of the past has been lifted from architecture: Renaissance is no longer its aim. We are marching to a dawn; but it is not of the long-set sun of classical precedent—it is the light of an entirely new science of building, that glorious adventure is to be sought.

This is an age of analysis and synthesis. We are constantly analyzing material substances, breaking them up into their original elements, and reproducing the original substances in the laboratory by a synthetic process —by gathering similar elements and combining them. In this process, we may, by changing the proportions or adding new elements, procure, in the synthetic product, new qualities or augment such qualities as we wish accentuated for specific reasons.

The production of cast stone, sometimes called concrete stone, "cast cement or cement stone," although not strictly a synthetic process, is very closely allied to it. For instance, we take natural stone, pulverize it, and treat it to produce cement. We mix this cement with aggregate—crushed stone of any kind, gravel, crushed shells, sand, and an almost infinite variety of materials—add water, and cast this plastic mixture in molds, or model and mold it by hand. On hardening, this mixture becomes an object of stone whose color and form might have been varied at will, which may be much harder and denser than natural stone, and the surface of which may be treated first, as natural stone, and in many other ways besides. This product is cast stone—not an imitation, but a synthetic product. We began with natural stone, broke it up, added the quality of temporary elasticity, making it possible to control its shape with very little labor, and also added the possibilities of controlling the color and surface texture.

By virtue of these added qualities alone, temporary plasticity, color and texture control, cast stone is a product worthy of the respect of the architect. Furthermore, there are the considerations of economy and quickness plus the convenience of production. It is no longer necessary to continue the practice of transporting large uncut blocks of stone, nor even the cut and finished product, long distances, as is so frequently done.

How long will it take the architectural profession fully to learn and then to teach the public that its art is at its best only when wholly sincere?

Today concrete is emerging from its era of disguise and concealment. Just why it was so long treated as something to be ashamed of is difficult to explain. When it attained structural reliability it became as respecta-

ble a material as any other, and it held a wonderful capacity for varied and attractive treatment. Yet for years designers concealed it expensively or dressed it like cut stone to the deception of none but the near-sighted. Its finish was but little less foolish than the "tin ashlar" that once adorned the edifices raised by ambitious citizens of our western boom towns.

The emergence of concrete has been gradual. Even yet its free and complete acceptance attracts attention, as in the case of the Brooklyn Army Base. We should have progressed beyond this point and, in expressing appreciation of that fine piece of work, it should not be necessary to compliment the designer upon his honest use of concrete.

Such use does not require that every member be displayed any more than that every joist or stud be visible in a frame building. But it does require that there shall be no subterfuge nor denial of the materials and, happily, we are rapidly approaching that stage.

Like the solid rock and the granite and sandstones of the Nile, concrete will, in due course of time, be shaped, formed and colored to meet the æsthetic requirements of the age in which it plays so important a part.

And so, when it is thus shaped and adjusted, and the building in which it plays a part is completed in rhythm with itself, we shall have a new phase of the building art. And then, if besides a new material, a new philosophy, a new and different basis of thought is ripening in the world, we shall find these other new elements asserting themselves in the buildings.

The Architectural Uses of Concrete.

By Irving K. Pond.*

Some few years ago, as chairman of a committee of the American Institute of Architects, I submitted a report on the uses of concrete in architecture which met with favor and was published separately by the Institute and widely distributed. I refer to that report as it was responsible for my giving a paper along lines suggested therein before a convention of this organization shortly after its appearance. I have only a vague recollection at this time of what was contained in either the report or the paper; only a detail here and there of either standing out in my memory. I am certain no one in this audience remembers as much as I do of the content of the paper, not even be he one of the few who heard it. For, as I recollect, on the afternoon when it was presented there was an excursion to Mooseheart and every delegate who could get away went on that excursion; so that there remained for an audience only those who of necessity were conducting the meeting and those others who themselves had papers to present. Of course the papers were printed in the proceedings; but I imagine they are few, besides the proofreaders, who scan the text of printed proceedings. So every one who appeared on that particular occasion might repeat his paper, quite certain that it would be "new stuff" to this audience. When the proceedings of this convention are printed anyone so minded can compare the records to ascertain if, unlike Shakespeare, the present authors have repeated themselves.

While in repeating verbatim whatever I may have said heretofore I would be showing scant courtesy to my hearers and would lower myself in my own estimation, I am frank to say that I shall deal now, as I dealt before, in great measure in generalities, leaving the specialists to deal with details both as to manufacture and application. Basic in all design and construction, and in the application of methods and materials, are broad general principles which should be understood and acted upon if there is to be a permanent advance in the building arts. It is with one or more of these principles that I am concerned in this paper which will touch upon details only as they may be made to clarify the general proposition.

The material which acts as a cohesive and binding element in this organization, the material which is common to all the products, the manufacture and application of which brings this body together, is cement; a

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material procured by mechanical processes from natural or from artificially produced rock, clay, or mineral elements. With cement as a binder the choice of an indefinite number of other materials is presented to form what is known as the aggregate; so that according to the will of the producer bodies hard, soft, smooth, rough, in monotone or rich in color, pervious or impervious to moisture, defiant of the action of fire or of frost, are forthcoming for employment in structures of a wide variety of use and beauty. Cement materials thus produced form the basis, according to the required characteristics, of structure, of exterior or of interior walls, of floors, of ceilings, structural or ornamental, either or both. The field would seem limitless. With all this seemingly infinite variety in use and potentiality one would think that there might come some originality of expression; one would expect something intrinsic in the material itself, something of an inner nature, which might be demonstrated upon the surface; something so *sui generis* that even a designer with but a superficial mental and spiritual equipment would hesitate to use it in imitation of other materials, or to leave the marks of other materials upon its surface. The possibilities of texture, the possibilities of color inhering in the product make it a thing through which the sensitive designer can make his feelings flow. So that to have this product made the medium of a wonderfully expressive art all we would seem to need is a wonderfully sensitive designer! But wonderfully sensitive designers do not grow upon every architectural bush; that is, wonderfully sensitive designers who feel the power and the rhythmic flow of structure.

It is especially with aesthetic possibilities inhering in structure that I, personally, am most deeply concerned and, therefore, it is with the potentialities of concrete in this rich field that I wish particularly to deal in this paper and so without further to-do I am going to plunge into, what to me is, one of the most fascinating and soul-searching of the psychological abstractions surrounding building in general and treat it in its especial relation to concrete as a medium of expression as well as of construction.

There are two methods of introducing concrete into structure; one is by setting precast units, as stone is set, and the other is by pouring the material in liquid or semi-liquid form into molds which characterize and define the form. The first method may be enlarged upon to a degree and in a manner to put it away beyond the simple placing of structural unit upon structural unit with the idea of producing a complete and harmonious whole. One ingenious writer upon what he terms the "super-city" takes his reader down an avenue in the making and shows him gigantic trucks laden with the precast sides of rooms with exterior and interior surfaces completely finished and decorated; and slabs with ceilings below and floors above ready to be lifted up and set in place by huge derricks when the preliminary work of setting up the pre-prepared walls shall have been accomplished. Whole towns are to be built as we when children used to build our card houses; except that deft powerful machinery is to take the

place of our clumsy little childish fingers, and great elaborately fashioned slabs of concrete are to stand in the stead of cards. "Beautiful" ball rooms for the restless, "beautiful" theatres for the tired, "beautiful" dining rooms for the hungry public; a public which need no longer hunger for "beauty"—for "beauty" will be met at every turn! Some cement association must have put the idea into the author's head. All that is needed to complete the picture are concrete men with natures capable of sympathetic acceptance of the concrete mass. The city may be traversed from end to end through freight and service tunnels in the basements; and, too, upon the house tops which will furnish landing stages for airplanes, parking places for baby carriages, and breathing spaces for all, above the dust and grime of the ground level streets; though without doubt some element will be found, if it does not now exist, which, combined with the mix or spread upon the surface, will eliminate the element of dust. This picture, not of my own creating but borrowed from an enthusiastic author who says the dream is about to be realized, is something for the concrete man,—not for him who is made of, but for him who works in, concrete,—to whet his imagination on. What a field of color and form is opened up to him; wherever his eye rests is "beauty!" But is it beauty? and if it is beauty how does it come about? or, better, how is it brought about? For the concrete mix does not of itself slump down into forms of beauty. Those forms are preconceived forms emanating from the hand and brain of man.

So we come face to face with a fundamental abstraction, a fundamental proposition of life. Life feeds upon life; beauty feeds upon beauty. There can be no life unless life is given to it; there can be no beauty unless beauty be sacrificed in the making. A God is crucified and buried that a God may be born. The grain is cut down and the seed buried in the earth that a new harvest may come forth. The masonry arch which spans in airy grace the rushing torrent was built upon a centering which was just as beautiful in its own way and in its own way just as much a work of art, but which had to be destroyed that the arch of stone might function. Even more so does this proposition hold with concrete structures which are to function for use and beauty. If the concrete product is to be perfect the form or mold must be perfect. That the concrete may be perfect the forms must be destroyed; that which was in itself perfect must be destroyed that ultimate and permanent perfection may be achieved. A poignancy as of grief must accompany the contemplation of the loss and destruction which must attend the creation and production of any structural object—the waste of hand power and brain power and heart power which must accompany the production of any object of structural beauty. leaving out of account the dreary hours of toil and drudgery in forests and mines and over weary trails that materials in the rough may be at hand, note a few of the elements of artistic craftsmanship which, only to be lost to the sight and knowledge of the general observer, must enter in that the exposed concrete of the structure may be a finished product;

carpentry and cabinet making, metal working and modeling. Each of these is accompanied by work in other trades. To my way of thinking it takes a deal of moral stamina for a carpenter or a craftsman in wood, or for a sculptor working in the clay, to carry out conscientiously a piece of work which he knows is to be destroyed that some other and less worthy material may come to exist in perfection of form—that his work must be lost that some other may come into being. The designer in concrete must be made to realize, if he does not, how much he owes to other arts and crafts and consequently how much he owes it to himself to treat material for which he is designing with respect and bring out its intrinsic potentialities to the best of his ability. He is not doing this if either in surface treatment or in structural form he imitates the characteristics of another material. If concrete is to be used it must be treated on its merits and handled with respect.

We have, in this country, developed a wide variety of textures and surfaces which may be applied to walls to make them look like concrete, though the background may be flimsy lath or clumsy bats of brick. We have in small measure as yet developed the constructional side as has been done in other lands, particularly in Germany and France. Our structure, outside of certain massive engineering works, has been confined to post and beam types, though arcuated structures are in evidence. But we have not spanned great spaces with structural vaults and ribs which take of themselves pleasing forms while following and containing the paths of structural forces, as has been done abroad. This is a field in which the structural imagination may well be allowed to play; a field in which our designers, with their material instinct for bigness and spiritual instinct for simplicity, should be eminently successful. Of course the interplay of stress and strain in many structural exigencies is not simple and if left unresolved would lead to an involved pattern which would not satisfy our desire for simplicity or directness of statement. But it is a part of the problem of the designer so to resolve the forces, so to direct the stresses, that unity shall appear and that an ordered simplicity shall characterize the design.

In the Old World permanency is a characteristic of structure generally; a characteristic sought to be obtained and expressed in concrete as in other structural materials. Where civilization is old a certain poise has entered men's minds; and haste, even speed, is not considered to be so vital an element. With us there is feverish haste and it evidences itself in our design. The pioneering spirit is still strong within us and we raise structures which will serve our temporary needs and which shall indeed be razed when we "go West" either in the literal or in the figurative sense. Our structures are leveled even by ourselves. Perhaps from the view of our developing art this is a happy state of things; for it gives us a chance to experiment without too great material loss. It gives us a chance to try out new forms and so the better to determine that which will best express us; will best exhibit our real character to ourselves;

best help us to develop that character. But that character, if it be character, is not to be developed by easy processes; it is to be achieved through process of stress and strain; through process of work done, well done, and obliterated; through scaffoldings built and demolished; through worthy forms constructed and destroyed. Our national art is to be achieved as our objects of individual art must come—through death and disintegration that life may be beautiful and abundant. If that which must ultimately be destroyed be substantially of a permanent nature, such as structures in concrete necessarily are, it is all the more necessary and desirable that we design with due regard to the characteristics of a material which lastingly presents its forms before the eyes of our impressionable minds.

This idea that I have been elaborating is beautifully expressed in one of William Watson's art maxims:

"No record Art keeps
Of her travail and throes,
There is toil on the steeps,—
On the summit repose."

Concrete may well be made the basis of the design; not simply the skeleton to support and to be filled in by other materials. I am just as conscious of the difficulties and just as keen as to the possibilities now as when I wrote that report years ago. Time alone will develop all the potentialities. Not time alone, but time aided by creators of processes, inventors of methods, and designers who are sensitive to the nature of the material. It is a long road and the end it not yet. But there is no need to be pessimistic, in spite of other words of William Watson's which I quote. Watson, again, is speaking of art:

"The thousand painful steps at last are trod,
At last the Temple's difficult door we win;
But perfect on his pedestal, the god
Freezes us hopeless when we enter in."

We need not worry seriously just yet as to that hopeless chill; for we have a long, long period of struggle in the sunshine of the concrete before we reach that difficult temple door of the abstract.

DISCUSSION.

Mr. Lindstrom.

ROBERT SETH LINDSTROM.—In the subjects that have been set forth, we have been talking about the splendid future of reinforced concrete. For the past nine years I have not, personally, been practicing architecture, but I have come in contact with many of the architects in and outside of Chicago, and I am going to say a few things to the architects tonight, about simplifying their practice and their specifications to meet the future needs of reinforced-concrete construction. In the first place, when I am called into offices to look over concrete specifications, having had some experience in the field and in my practice, I find in a number of specifications when we come to concrete ingredients such as portland cement, aggregates and water that in some specifications I find the name of a portland cement that has been extinct for at least fifteen years. I also find four or five pages regarding the manufacture of portland cement, and I find one small paragraph saying, "The sand shall be clean, sharp sand."

The American Concrete Institute and the contractors in the field of reinforced concrete have learned that the aggregates and the sand are of more importance than the particular brand of portland cement, and if you want to simplify your practice and get better prices and more uniform prices on your buildings, then you specify that cement shall be portland cement in accordance with the A. S. T. M. specification, and mention no brand, then you, the architect, the contractor, and the manufacturers will get along better. Again, specify the use of Fuller's formula for sands, which today meets with the sand termed torpedo sand in the Chicago market. One exception to this is necessary—add that 5 per cent of the fine sand shall pass a 100-mesh sieve. As to the water ratio, you already have a lot of knowledge on that, and if you will take these into consideration with the grading of your aggregates, your future problems in concrete construction will be simplified.

The Behavior of Engineering Structures in Recent Violent Wind Storms.*

By Roy L. Peck and Norman M. Stineman.

Two wind storms of major intensity have occurred in densely settled portions of North America within a period of nineteen months. These storms left individual records of destruction and stability of structures which have contributed immeasurably to our knowledge of building. Collectively, they permit us to draw certain conclusions which should be of material assistance to engineers and architects who must design structures which may be subjected to similar violent disturbances.

The main features of interest in these storms and their effect on engineering structures are summarized in the reports which follow together with certain conclusions representing the lesson from these two experiences. For the sake of continuity the reports are presented in chronological order.

* Reports and conclusions on the Florida Hurricane of Sept. 18, 1926, and the Cuban Hurricane of Oct. 20, 1926.

EFFECT OF THE FLORIDA HURRICANE ON STRUCTURES.

By Roy L. Peck.

General Description of the Hurricane.—Striking Miami, Florida, and the adjacent territory, the most devastating hurricane on government records began its northwesterly sweep across the state on Sept. 18, 1926. In its wake were hundreds of dead, thousands of injured, and a trail of destruction that staggered the entire world.

Property damage over the whole area affected has been placed after surveys by relief officials, at approximately $75,000,000, and the destruction in the Miami district alone at about $50,000,000. The storm-affected area on the East Coast extended 60 miles north and south, reaching inland to the Everglades, and included the municipalities of Miami, Miami Beach, Coral Gables, Hialeah, Hollywood, Dania, Ft. Lauderdale and the intervening territory between Pompano and Florida City.

According to the U. S. Weather Bureau, at Miami, between 8 and 15 in. of rain fell during the 7-hr. period of the two storms, the first blowing from the northeast, the second, and heaviest, from the southeast. A large part of the water in the air was sea water, picked up by the wind from the ocean and Biscayne Bay and carried inland. This water-laden air, borne by a wind with an estimated velocity of 125 miles per hour, with accompanying gusts of much greater intensity, caused much damage to all types of structures in which proper construction methods had been slighted or wholly forgotten when the buildings had been built.

The storm possessed typical characteristics of a tropical hurricane, in travelling in "lanes of varying intensity" which were parallel to its general course. Such performance resulted in maximum and minimum effects on similarly built structures located adjacently throughout the path of the storm and makes it difficult to draw accurate conclusions on the relative resistance to the wind force of different types of construction and materials.

Summary of Damage.—Approximately 5,000 houses and buildings were completely destroyed and perhaps double that number suffered some damage in the area of about 120 sq. mi. within the limits above stated and extending back from the Coast from two to ten miles. All types of construction and classes of materials have been employed in buildings in this area, with the exception of brick and stone. The use of these latter types is so small that they cannot fairly be included in the consideration of the suitability of the various types to withstand high wind pressures.

The relative quality of the construction itself in every building affected determined largely its resistance to damage. When well built, structures of all types withstood without damage wind pressures from the water-laden air many times that which they had been designed to withstand.

The construction in Coral Gables is an outstanding example of this, as in this city less than 3 per cent of the buildings and residences were badly damaged and only 8 per cent sustained damage of any description other than superficial loss of awnings, etc.

A general survey of the first reports and pictures from the affected area, broadcast from many sources, might have indicated an almost complete destruction. Actually, however, from this confusing mass of evidence, a careful study has revealed certain well defined and typical in-

FIG. 1.—VENETIAN HOTEL, MIAMI, FLORIDA.

This is a ten-story structure of reinforced-concrete skeleton frame with tile walls. It was built in 1925 in strict accordance with the building code of Miami and was undamaged though located at an especially exposed point.

stances of damage to structures which, when combined with statements of witnesses as to the sequence of the failure in any case, permit us to draw valuable conclusions.

TYPICAL INSTANCES OF DAMAGE TO STRUCTURES.

1. *Skeleton Frame Buildings.*—Large buildings of the skeleton frame type with rigid frames of reinforced concrete or well-braced structural steel suffered no structural damage. Out of perhaps 25 high buildings of this type in Miami, only two were damaged considerably. These were the Meyer-Kiser Bank Building and the Miami Realty Board Building. Both

were unusually narrow in plan and were so situated as to be openly exposed to the wind force from all directions. The Huntington Building, located on S. E. 1st St. and S. E. 2nd Ave., Miami, similarly constructed, suffered slight cracking in the walls and in the partitions in lower floors, probably due to the swaying of this structure in the wind.

The Meyer-Kiser Bank Building, facing south on First Street, is a 17-story steel frame structure, 45 ft. wide by 140 ft. deep, with 8-in. hollow clay tile walls and fireproofing, clay tile and concrete joist floors, and 4 to 6-in. gypsum block partition walls. The front exterior is of sand-lime face brick, the remainder of the building being finished in stucco applied directly to the clay tile.

FIG. 2.—MIAMI-BILTMORE HOTEL, CORAL GABLES, FLORIDA.

An eleven-story reinforced-concrete structure with a nineteen-story tower of steel frame construction encased in concrete. The only damage consisted of the loss of some roofing tile, windows and awnings.

A summary of the damage to this building is as follows: A severe distortion of the building laterally at the front (south) wall, the structure leaning to the west from the 3rd to the 10th floors, being about 30 in. out of plumb at this floor. A reverse bend occurs here, the remaining seven stories being practically vertical. The building also leans to the north, being bent longitudinally, with the top about 8 in. from the vertical. Exterior walls on the front face from the 4th to the 11th floors are collapsed, leaving the frame entirely exposed. Large cracks are opened up along the side (east and west) walls, with lesser cracking on the rear (north) wall. The interior of the building is severely damaged, most of the east-west partition walls between the above floors in the front portion

being collapsed and the north-south partitions badly cracked. No cracks are to be seen in any of the ceilings or floors in the building. The 24-in. girders in the front wall are straight and horizontal, the 12 to 14-in. compound section columns being bent sharply just above and below the girders, with the connection angles being badly distorted and some cracked. The building was designed for a 20-lb. wind pressure in accordance with the Miami building ordinance, which, however, permits wind stress to be disregarded where it does not exceed 50 per cent of the combined live- and dead-load stress; and where it does exceed that amount the working stress used in computing the structural section may be 50 per cent higher than

FIG. 3.—LARGE APARTMENT BUILDING, CORAL GABLES, FLORIDA.

Minor damage to roofing tile and awnings, with water damage to furnishings caused by the leaking roof were the after-effects of the storm in Coral Gables, where good construction proved itself.

ordinary. Wind bracing consisted of knee brackets between the columns and girders up to the 3rd floor in the front and rear walls and some additional bracket stiffening on the rear wall.

The Miami Realty Board Building, facing west on N. E. First Ave., is a 15-story structure, 45 ft. wide by 95 ft. deep, of practically the same construction as the Meyer-Kiser Bank Building, but with the whole exterior finish being of stucco plastered on the clay tile walls. The building is cracked quite badly in the walls of the lower four or five stories, especially in the narrow face at the rear (east) where the stucco exterior has spalled extensively along the lines of the floor girders up to the 8th floor. The side walls are also cracked and bulged at several points in the lower stories.

FIG. 4.—MEYER-KISER BLDG., MIAMI.

Shows damage done to the building in the lower stories, on the southeast corner. The construction is of steel frame, seventeen stories high, with fireproofing and filler walls of clay tile and brick. The building is badly distorted, much of the front wall destroyed, with severe cracking of interior partitions and longitudinal exterior walls. Condemned by the Building Department, the building will be demolished to the eighth floor to permit examination of its true condition.

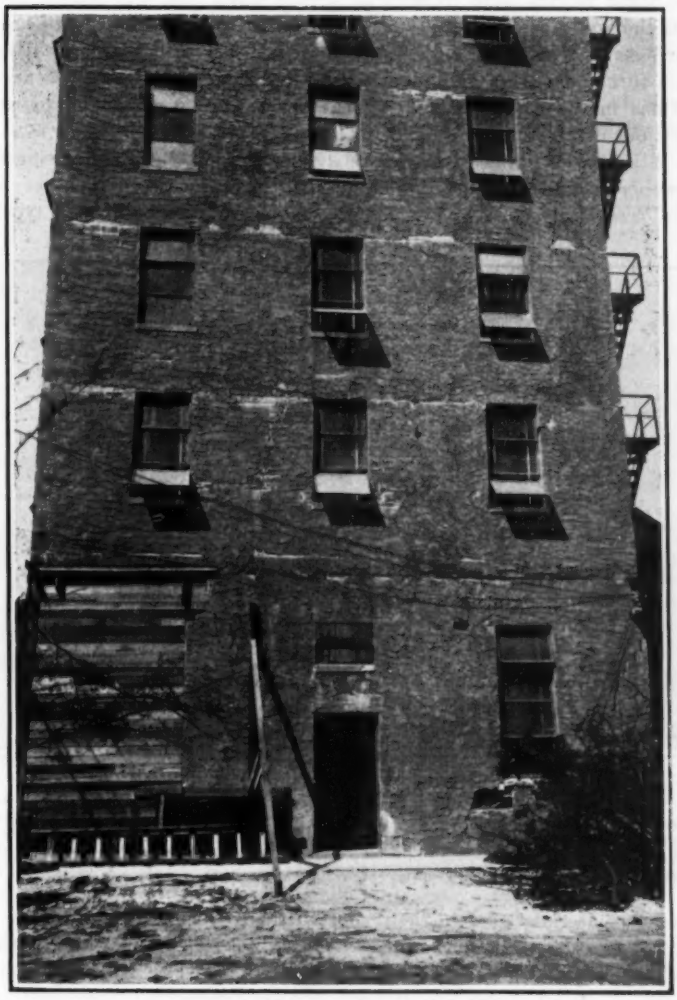

FIG. 5.—MIAMI REALTY BOARD BUILDING, 1ST AVE. AND 5TH ST., MIAMI.

A steel frame structure of fifteen stories with hollow clay tile fireproofing and stucco exterior finish. This building bent under the force of the wind until the elastic limit of the steel frame was almost reached but not exceeded, since no permanent deformation of the frame exists. This view is of the rear (east) wall and shows how the filler walls have cracked at the girder lines in the lower six stories under the bending action.

On the interior, a number of transverse and longitudinal partitions in the first three stories were cracked or broken down and the plaster spalled from the walls in the front (west) portion of the building up to the 6th floor. As in the Meyer-Kiser Bank, no cracks are to be seen in ceilings or floors throughout this building. Lateral bracing is provided by beam connections in the front and rear walls. The building was found to be but 5/8 in. out of plumb.

2. *Large Buildings with Unpartitioned Areas.*—Buildings having large unsupported or unpartitioned areas, such as garages, churches or halls, quite frequently suffered complete collapse, regardless of the types of

FIG. 6.—HIALEAH FRONTON BUILDING, HIALEAH, FLORIDA.

Built with monolithic concrete walls, and steel frame roof trusses supporting a long wood roof. The debris in the foreground shows the collapsed condition of the wood frame pavilion. No damage occurred to the walls of the building, although they presented a large area to the wind, and were totally unprotected by adjacent structures.

masonry materials used in their construction. Such structures have been referred to as reinforced-concrete structures in some accounts of the damage due to this hurricane. This is not an accurate classification, for while reinforced-concrete columns, pilasters or beams were frequently used in the walls, there were no cross beams of reinforced concrete forming a frame for the whole building. They were in reality of ordinary construction of two stories of concrete block construction with flat or low hip wood roofs, and high parapet walls. According to statements of witnesses and from inspection of the ruins of many such structures, it appears that the parapets were first blown over, either onto the roof or to one side. This opened up an entry for the wind either through the damaged roof or along the

opened roof joist line of the wall. The roof or walls were then dislodged by the air pressure which had been built up inside in a balloon action by the wind which lacked vent on the far side of the building. The subsequent fall of the roof removed the lateral support it had given the walls, which in turn were blown down as a result.

In the few instances where such buildings were constructed of wood, as in cafés and pavilions of large size, the roof trusses first broke beneath the pressure, or the roof was blown off wholly or partially, followed by a

FIG. 7.—BILTMORE THEATRE, 143 N. E. 40TH ST., MIAMI.

Illustrates the non-resistance to a high wind force of a masonry wall built with weak mortar, and not properly supported by pilasters and reinforced-concrete frame. Also lacking tying-in of roof joists to the walls and frame cross-ties. Buildings of this type of construction suffered severely.

collapse of the walls. Other such buildings gave way as a unit before the wind pressure, suffering considerable distortion of the whole structure without collapse.

3. *Apartment Houses.*—A constantly recurrent type of construction and its accompanying failure so often observed as to warrant separate mention here was that construction and damage seen in numerous two-story apartments where good construction had been slighted. Concrete block of a fair grade usually formed the walls of such buildings. No structural frame, either of reinforced concrete or structural steel, had been used, and the masonry had been laid up with thin joists of very weak

FIG. 8.—MANCHESTER APARTMENT BLDG., 55TH ST., N. E. MIAMI.

An instance of failure of high parapet walls and poorly constructed roof resulting in the side wall, lacking pilasters, being blown down by the wind which had entered the building. Note condition of front parapet wall and roof construction.

FIG. 9.—RESIDENCE AT 165 17TH AVE., HOLLYWOOD, FLORIDA.

View to the northeast. This house was of wood frame without any diagonal bracing and in which very light construction had been used. The wind pressure was sufficient to crush this with a consequent tilting of the whole structure, as shown.

FIG. 10.—RESIDENCE, CORAL GABLES, FLORIDA.

The damage to natural growth indicates the force of the wind in this region. The residence, built of concrete masonry, was undamaged. The Miami-Biltmore Hotel is in the left background.

FIG. 11.—GARAGE, 1ST AVE. AND 14TH ST., S. W., MIAMI, FLORIDA.

The initial cause of damage here was the failure of the wood roof trusses which were anchored very lightly to the pilasters and were of inadequate size for their span length.

mortar. Generally, no adequate tie was used between the floor joists and the walls nor were pilasters provided. In the few instances where pilasters had been used in the walls below, these had not been run up into the parapet walls above the roof. This use of high parapets is in accordance with the prevailing architectural style of the region. Flat roofs, constructed of light timbers covered with tar paper or composition roofing,

FIG. 12.—ELEVATED WATER TANK, COCOANUT GROVE, MIAMI, WITH REINFORCED-CONCRETE SUPPORTING STRUCTURE.

Although located at an especially exposed point of the beach and sustaining heavy water as well as wind impact, the structure is plumb and without cracks, indicating that its design, construction and foundations were satisfactory. Superficial damage consisted of loss of roof of tank and dislodgment of the ladder.

completed the "boom period" construction. The sequence of the failure was as follows: The high parapet walls were first blown over, their fall damaging the roof or opening up and otherwise weakening the wall along the roof joist line. In no cases where damage was observed had any effort been made to anchor or tie the roof and floor joists into the wall, other than to set these in between the blocks in a bed of the weak mortar. In the absence of any tie support from the floors or roof, and lacing strength, the

wall was often blown out for a story height or more. In a majority of cases where rebuilding was in progress, the old blocks were again being used as it was quite simple to remove from them the traces of crumbly mortar which remained. The block units themselves were invariably intact.

4. *Residences.*—Of all structures in the area affected, residences, quite naturally, suffered most heavily. There were, perhaps, approximately 40,000 dwellings in this area. According to the best available final estimates, one-eighth of this number were completely destroyed and one-fourth damaged more or less seriously. Probably 60 per cent of all the houses

FIG. 13.—MAXWELL ARCADE BUILDING, FT. LAUDERDALE, FLORIDA.

This shows the destruction of a three-story high electric sign on the roof. The building, of reinforced-concrete, was undamaged structurally, though plate glass windows on the ground floor were destroyed. The condition of the sign is typical of many such losses, although these were supposed to have been built with the same wind load protection as buildings and with proper supervision of their anchorage to the building.

were of masonry and 40 per cent of wood frame construction. Of those built of masonry, probably 90 per cent were of concrete block construction. The preponderance of this type has caused newspaper and magazine reports to over-estimates the failures of concrete masonry.

When well-built, residences of all types stood up without damage except the destruction of awnings, window glass and decorative appurtenances. These included houses with well-braced wood frames of either stucco or clapboard exterior, having adequate anchorage to their foundations, concrete block houses with good mortar joints, plisater strengthening and sufficient tying-in of roof and floor joists to the wall to form a struc-

tural frame, and houses of any other type where the principles of good construction in that type had been adhered to. All "boom period" built residences which were cheaply constructed suffered accordingly. This rule was everywhere unbroken.

Roof loss on these types was by far the greatest single item. Usually the second story windows blew in or were otherwise broken and the increased pressure which was built up within lifted off the roof in whole or part. This weakened the resistance of the already weak walls which usually were blown out on the leeward side of the house. Anchorage of the structure to the foundation, in the case of frame residences, was quite frequently forgotten and resulted in bodily movement, overturning or other dislodgement of the house if the frame construction itself had been strong enough to prevent collapse of the whole structure.

5. *Roofing Materials.*—The failures of roofing materials in general was large and important, as these were responsible for much more extensive damage to the structure beneath when a ready entry for the wind and rain had been opened up. The knife-like action of the wind made use of the smallest opening to get inside the structure and build up the pressure to dangerous proportions. Sheet roofing, such as tar paper, if nailed down, usually ripped away, but if this had been cemented down, it usually resisted the clawing action of the wind much better. Mission or Spanish tile roofs, either of clay or concrete, had the convex tiles blown off in many cases. Interlocking roofing tile came through more intact. Tin roofs were invariably blown off. Much damage was done to roofs by heavy flying missiles, especially in the downtown district where roofing tile blown off high buildings attained tremendous velocities and striking force.

6. *Miscellaneous Damage.*—Elevated tanks and water towers in general withstood the storm, although the roofs of several tanks were blown off and the ladders dislodged. Window glass in all kinds of buildings was invariably blown in or broken on the eastern and southern exposures. Two brick chimneys and a steel stack, each about 150 ft. in height, were blown over. The five 450-ft. steel radio towers of the Tropical Radio Telegraph Co. at Hialeah were blown down. Nearly 100 miles of transmission lines were leveled with their supporting wood poles. Overhanging and roof signs were blown over almost without exception. Light standards, both of concrete and cast iron, suffered breakage in all locations. Scaffolding, formwork, hoisting towers and loose objects around buildings under construction were invariably destroyed.

SUMMARY AND CONCLUSIONS.

1. As regards tall buildings of the skeleton type, the outstanding fact to be noted from this study is that the damage was very limited.

2. For such buildings of ordinary proportions, the lateral rigidity was such that no damage occurred even under wind pressures of hurricane intensity.

3. The three buildings where structural damage resulted out of 20 or more of this type, were extremely narrow in proportion to their height and were so situated as to receive the full intensity of the storm.

4. In these three cases cracking of the walls and fireproofing resulted from the deflection of the structure, which cracking in one case was sufficient to reduce the stiffness to the extent that a permanent distortion in the frame ensued.

5. Apparently, the 20-lb. wind loading used in the design (modified as permitted by the Miami building code) was not sufficient for winds of hurricane intensity in structures of such exposure and dimensions. The probability is that much of the damage resulted from a swaying or rocking of the structures under the pulsations of the wind.

6. Large buildings of masonry with unpartitioned areas, such as garages, churches or halls, should have some system of cross-beams securely fastened to bearing walls, pilasters, or columns, forming a rigid frame to stiffen the structure as a whole and laterally support the exterior walls.

7. In general, buildings of masonry built in accordance with the "Recommended Minimum Requirements for Masonry Wall Construction," of the U. S. Department of Commerce, were not damaged structurally.

8. Wooden shutters constitute an excellent safeguard against window loss in residences, which in many instances was the start of the destruction of the entire house.

9. In wood-framed structures, some form of adequate anchorage to foundations should be provided, and care taken during construction to see that all rafters and ends of partitions are securely spiked to the sidewalls and that diagonal bracing is used.

10. Overhanging or roof signs should be designed for the same wind loading as buildings and the method of anchoring such signs to structures should be passed upon by the building department.

11. Miscellaneous engineering structures, such as elevated tanks, water towers, etc., withstood the wind with no damage, indicating the sufficiency of the usual engineering design of such structures.

12. Pulsations of force from gusts of wind should be considered as supplying maximum wind intensities and contributing most toward the damage done to structures, especially when the movements of the structure were synchronized with the intensive gusts of wind.

13. All materials when in good construction resisted damage and when in poor construction failed.

THE CUBAN HURRICANE.

Oct. 20, 1926.

By Norman M. Stineman.

The Storm.—The violent hurricane that devastated western Cuba and the Isle of Pines on Oct. 20, 1926, passed in a general direction from southwest to northeast, with the center of the disturbance passing about 15 miles southeast of Havana. The wind blew with hurricane velocity for about twelve hours, continuing from about 2 o'clock in the early morning of Oct. 20 to about 2 o'clock in the afternoon. At first the wind blew from the east, but at 5 o'clock in the morning it shifted to the northeast. After 10 o'clock in the forenoon there was a sudden shift to the north, and still later to the northwest.

Velocity of Wind.—The heaviest blow occurred from about 9.30 to about 11.30 in the forenoon, but before that time the aerometers at the National Observatory had been blown away, at the instant when they registered 96 miles an hour. The velocity grew considerably higher after the destruction of the aerometers. Two days later, when the same storm struck the Bermudas, the official weather bureau gage is said to have registered 114 miles an hour when it was blown away. The consensus is that the Cuban storm reached a velocity of from 120 to 130 miles an hour and that it was of a severity equal to that of the Florida hurricane of Sept. 18, 1926.

Probable Force of Wind.—The probable force of the wind, expressed in pounds per square foot, can be estimated only after analyzing the theoretical lateral resistance of the structural frames of buildings which were in exposed locations. A number of modern buildings in Havana, ranging in height from 7 to 10 stories, are surrounded by older buildings of two or three stories. The taller modern structures were consequently subjected to the force of the wind for nearly their full height. Inasmuch as no damage occurred to any of these buildings, an analysis based on the ultimate strength of the structural frame might give us some idea of the force of the wind.

Not all buildings in Havana are designed for the same assumed wind pressure. One large firm of engineers designs against an assumed pressure of 30 lb. per sq. ft.; but the city building regulations are not specific on this point, and the building department is known to have accepted designs that were computed for pressures less than 30 lb. Some impression of the force of the wind may be obtained from Fig. 14, showing a 2-in. x 4-in. timber which had completely penetrated the trunk of a royal palm.

Preparation for the Storm.—Warnings of the storm's approach were first announced on the morning of Oct. 18, and on the following morning the warnings were applied definitely to Havana and western Cuba. An-

nouncement was made that the hurricane might be expected shortly after midnight, in the early hours of the 20th. All day and evening on the 19th the people of Havana were busily preparing for the storm, boarding up plate glass windows, securing shutters on doors and windows, getting merchandise under cover, putting water craft in the safest places available, attaching guy wires to derricks and construction towers, and taking such

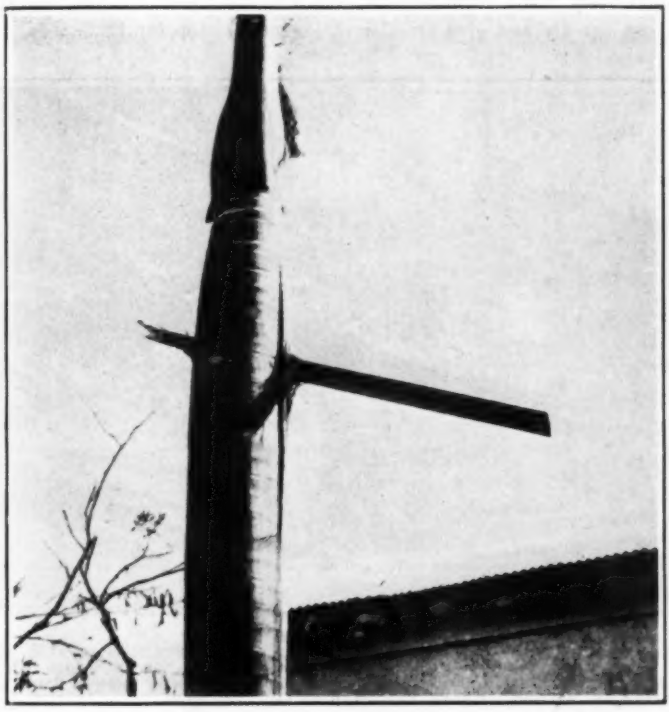

FIG. 14.—TIMBER BLOWN THROUGH ROYAL PALM.

The force of the hurricane that swept western Cuba on Oct. 20, 1926, is well illustrated by this picture of a 2-in. x 4-in. timber that was hurled against a royal palm in Batabano with such force that it penetrated the trunk of the palm. A curious coincidence is that a similar thing occurred during the tornado that swept Southern Illinois and Indiana in March, 1925.

other precautions as previous experience dictated. By midnight the city was fairly well prepared for the storm.

Loss of Life and Property.—The exact loss of life will probably never be known. Estimates place the loss at a figure approximating 500. Of this number about 55 were lost in Havana, not including the deaths in the harbor. The property loss is variously estimated at amounts approxi-

mating $100,000,000. The greatest loss of both life and property occurred in the smaller towns and villages and in the open country.

The general destruction of crops, trees and shipping was enormous, and the same is true of buildings in the smaller cities, towns and villages. The sugar crop appeared at first to have suffered heavily, but the canes recovered themselves and are producing virtually a normal crop. The sugar industry, however, suffered great loss because of the extensive destruction of sugar mills. The 1926 banama crop was destroyed, but the new stalks grew out rapidly and give promise of the usual crop for 1927.

FIG. 15.—PARTIAL DESTRUCTION OF WOODEN DOCKS, AND DAMAGE TO WOODEN DOCK BUILDINGS.

This was at point in harbor that served as a "pocket" into which much shipping craft were driven. This point is directly across from the Central Ry. station. Cuban hurricane of Oct. 20, 1926.

Damage to Shipping.—Between forty and fifty boats of various sizes were sunk or beached in Havana Harbor. Many others suffered serious damage from being tossed about, thrown against each other or crushed against concrete piers or wharves. The damage to shipping was intensified from the fact that when the storm blew the hardest the direction of the wind was toward and through the harbor entrance. Huge waves were driven into the throat of the harbor, as shown clearly in Fig. 20, taken on the afternoon of the storm. This action banked up the water in the harbor to a level such that low buildings along the lower parts of the waterfront were submerged almost to the second floor.

Boats of all sizes broke from their moorings and were driven about, and by the time the storm began to subside the utmost confusion prevailed.

Some of the harbor craft were thrown up on wharves and piers, some were still floating, while others were sunk and lying beneath those afloat.

Harbor Structures.—Harbor structures at Havana, such as docks and piers, are of two general types. The older structures are of wood, many of which were either damaged or partly destroyed. Several were almost completely destroyed. The newer type of waterfront structure, built in compliance with a federal law applying to all harbor work in Cuba, are supported on reinforced-concrete piles, over which are deck structures of reinforced concrete or of concrete-encased structural steel, with the former type predominating. The wharf buildings are of reinforced concrete.

FIG. 16.—HARBOR WRECKAGE THROWN UP ON THE WHARVES IN HAVANA HARBOR DURING THE CUBAN HURRICANE OF OCT. 20, 1926.

Among the newer type of harbor structures neither substructures nor superstructures suffered any damage.

Wooden structures on the docks and wharves were quite generally either damaged or destroyed. Most notable among them is the Atares Warehouse, owned by the American and Cuban Terminal Co. The main warehouse and the auxiliary buildings, all of which were of wooden construction, with corrugated iron roofing, were completely destroyed. Other damaged wooden harbor structures were found in many sections of the waterfront. (See Figs. 15 and 16.)

Among the reinforced-concrete pier and wharf structures, none of which suffered any damage, are the San Francisco Pier, the Machine Pier,

the Santa Clara Pier, the Ward Line Pier (Fig. 17), and terminal warehouses at the four piers just named. All these are splendid examples of reinforced-concrete construction, and show evidence of good design and supervision.

Chimneys.—Tall reinforced-concrete chimneys made a praiseworthy record in this storm, whether considered from the standpoint of their own performance or in comparison with the performance of tall brick chimneys and steel stacks.

In the entire area thirty-two tall concrete chimneys of the industrial type were not damaged in the slightest. Four concrete chimneys with very thin shells and light reinforcement, all designed and built by the same concern, were damaged. One of them, at the Toledo sugar mill, had its

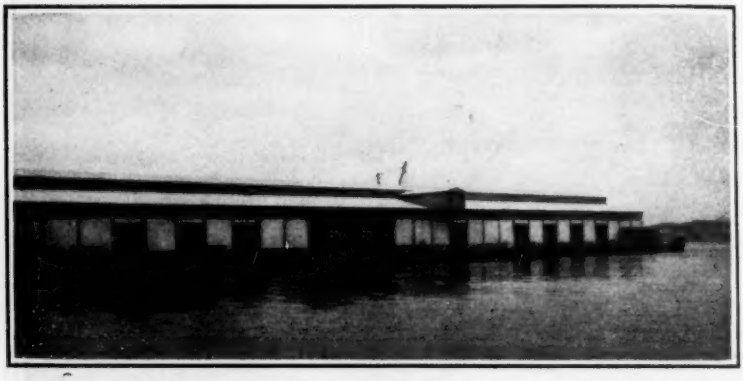

FIG. 17.—WARD LINE PIER, HAVANA.

One of the finest examples of reinforced-concrete waterfront construction in Havana harbor. It was this pier that laid up the 6,000-ton steamer, Barcelona, when the steamer crashed against the pier during the hurricane of Oct. 20, 1926. The pier did not suffer the slightest damage.

upper part blown off. The sugar mill itself was completely destroyed. Another one, at the Mercedita sugar mill, was badly damaged, while two at Gomez Mena were damaged to some extent. These four chimneys are the only failures or part failures among reinforced-concrete structures of any kind, in the entire storm area of Cuba.

Brick chimneys did not fare so well. An incomplete check-up disclosed eight large brick chimneys destroyed, one badly damaged and one slightly damaged, out of a total of not more than thirty chimneys of this type in the storm area.

Among steel stacks the number of failures was very large. No attempt was made to compile a record.

Excellent examples of reinforced-concrete chimneys are shown in Figs. 18, 19, 20, 21, 29 and 30.

FIG. 18.—VIEW OF A LARGE CLAY BRICK MANUFACTURING PLANT IN PUENTES GRANDES, A SUBURB OF HAVANA.

Note the general destruction of buildings, which were all blown down. The picture shows also the destruction of the company's two large brick chimneys, while the two reinforced-concrete chimneys remained standing and suffered no damage.

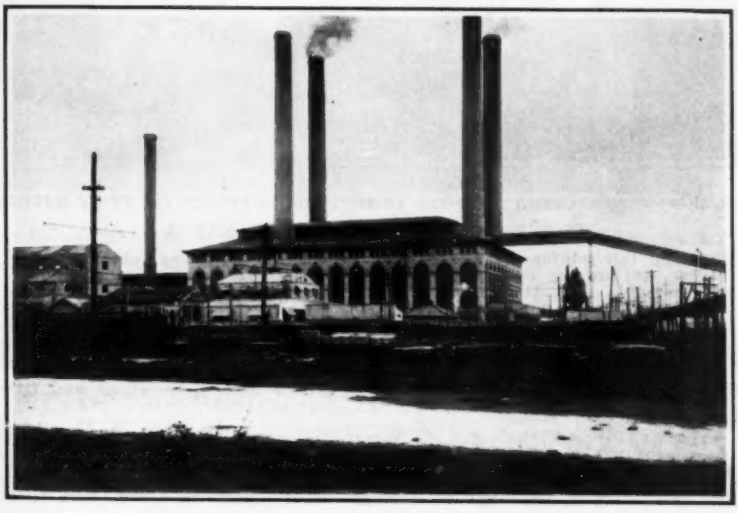

FIG. 19.—THE FIVE HUGE REINFORCED-CONCRETE CHIMNEYS OF THE HAVANA ELECTRIC LIGHT AND POWER CO.

Were not damaged to the slightest degree in the hurricane of Oct. 20, 1926. Designed and built by the Havana Agency of the Weber Chimney Co., undef the direction of William Croft, now head of the General Concrete Construction Co., of Cuba.

Quality of Concrete Work in Cuba.—Interviews with American and Cuban engineers in Havana indicate that concrete as a rule is of no more than fair quality, due to somewhat inferior aggregates. Sand and gravel are obtained largely from the ocean beach. Some river sand and gravel are used, but the river product sells at a higher price without producing much better results.

The Frederick Snare Corporation, one of the largest construction companies, conducted some tests from which they obtained the best results with a mixture that varied to some extent, but averaged about 4½ sacks

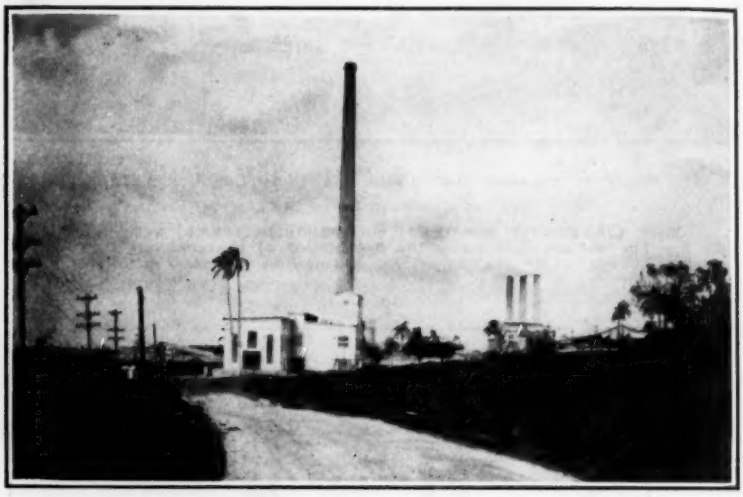

FIG. 20.—AT THE CUBAN PORTLAND CEMENT CORPORATION'S PLANT AT MARIEL, IN THE PROVINCE OF PINAR DEL RIO, CUBA.

The tall reinforced-concrete power house chimney was subjected to a severe test during the hurricane of Oct. 20, 1926. This is the tallest chimney in Cuba. It is 250 ft. high. The other three chimneys are over the kilns. They all weathered the storm without damage.

of cement, 6 cu. ft. of fine Terar white sand, 6 cu. ft. of quarry grits, and 16 cu. ft. of crushed stone, one-third of which was ¾-in. stone and two-thirds was 1½-in. stone.

Old and Modern Buildings.—Buildings in Havana and vicinity must, for the purpose of study, be separated into two general classes, namely, those built before the days of modern development in Cuba, and those built under the influence of that development.

The Havana sky-line has no resemblance to the sky-lines of large cities in the United States. Very few buildings extend above four or five stories. Of the modern reinforced-concrete and concrete-encased steel buildings, which vary in height up to ten stories, only four buildings (two of

each type) reach this latter height. (Fig. 22). There is now under construction a concrete-encased steel building for the Cuban Telephone Co. that will be twelve stories high, including a two-story tower.

Older Office and Mercantile Buildings.—Very few of the older office and business buildings are as much as five stories high, while most of them range from two to four stories. The Manzana de Gomez (Fig. 23), five stories high and covering one block, is probably the largest office building in Havana in the matter of floor space. Individual stories in these buildings, however, are much higher than in corresponding buildings

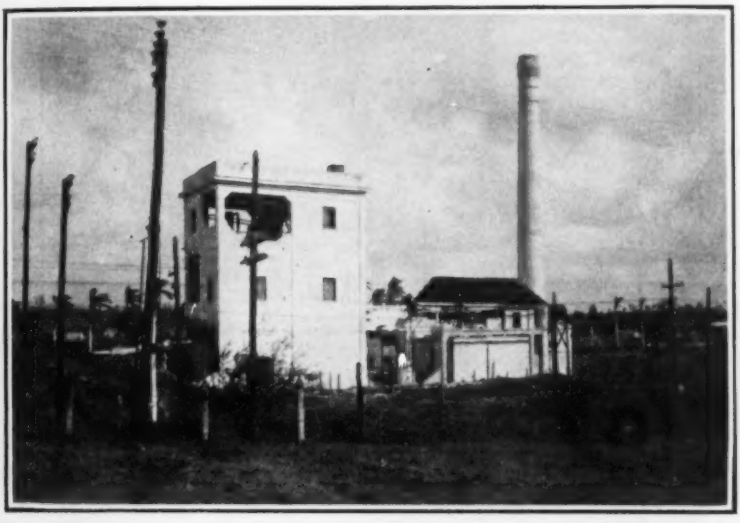

FIG. 21.—DESTILADORA MODERNA (MODERN DISTILLERY) AT RAILWAY TRACKS SOUTH OF BATISTA STATION, HAVANA.

This building has a reinforced-concrete structural frame, with filler walls of brick. Two of the brick filler walls were blown out, but the building suffered no other damage. The reinforced-concrete chimney likewise was undamaged. The brick building at the right had the roof and upper part of the walls blown off. Cuban hurricane of Oct. 20, 1926.

in the United States. For example, the third story of the Manzana de Gomez is 17 ft. from floor to ceiling. Even the modern buildings have much higher story heights than in the States. In the Seville-Biltmore Hotel, to cite one instance, the upper floors are 13 ft. and 3 in. from floor to ceiling. The purpose of the high ceilings is to permit circulation of air during hot weather. These older buildings are of very heavy construction, but not of particularly good workmanship. Their low and uniform height and their massiveness provide good resistance to storms.

Older Dwellings.—Most of the older dwellings, like the older office and mercantile buildings, are built of stone masonry, coated with stucco.

It may be said that all buildings in Havana are coated with stucco, for the number not so treated is negligible. This stucco, or plaster, formerly was made of a mixture of lime, clay and sand. The mixture now used is lime, portland cement and sand. A typical failure of a wall of stone masonry is shown in Fig. 24. An older and more primitive form of masonry consists of very thick walls made of a mixture of clay and lime,

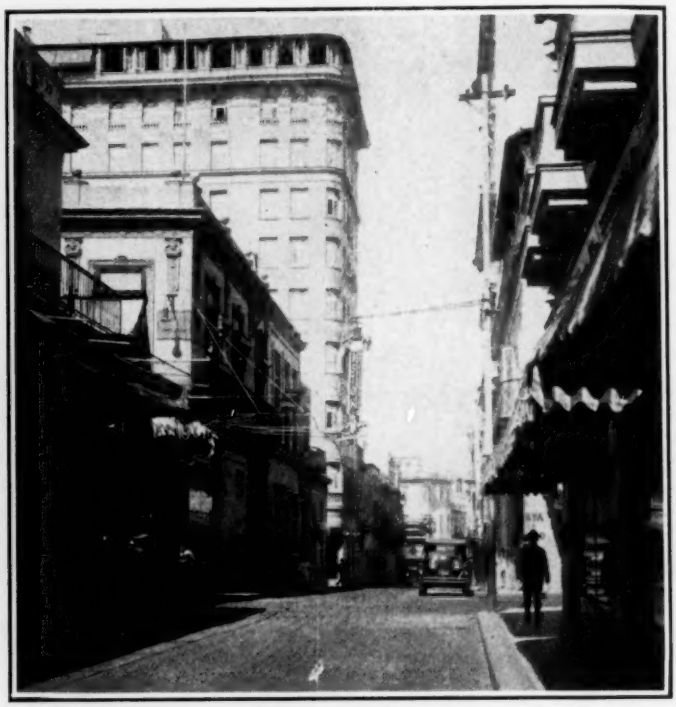

FIG. 22.—HOTEL ASTOR, AT THE NORTHWEST CORNER OF SAN MIGUEL AND AMISTAD STS., HAVANA.

Ten stories high, with a reinforced-concrete structural frame and floors, the exterior coated with portland cement stucco. Although this building is very narrow and the sides are exposed for nearly the full height, it suffered no damage in the hurricane of Oct. 20, 1926.

into which small stones were placed, after which both the exterior and the interior were plastered. It produces a wall of soft masonry, not strong, but it is heavy and provides good insulation against heat.

Flooring.—The prevailing floor surface in buildings of all types is tile, either clay or cement tile, the latter made in Havana by workmen well skilled in the trade.

Roofing Materials.—Flat roofs are surfaced with flat tile, and sloping roofs with Spanish tile. The latter were in a number of instances blown off during the hurricane, but the sheathing remained in place. There was comparatively little unroofing in the city except where entire sections of buildings were destroyed.

Buildings in Smaller Cities, Towns and Villages.—In the smaller places and in the open country the destruction of buildings was in far greater proportion than in Havana. The condition of these outlying com-

FIG. 23.—MANZANA DE GOMEZ, PROBABLY THE LARGEST OFFICE BUILDING IN HAVANA, 5 STORIES HIGH AND COVERING AN ENTIRE BLOCK.

This type of low, heavy masonry construction suffered little damage. The offices of the Cuban Portland Cement Corp. are on the third floor of this building. This photo was taken three weeks after the Cuban hurricane of Oct. 20, 1926.

munities was deplorable, for the destruction of buildings was widespread and little had been accomplished a month after the storm toward making repairs or rebuilding. The impoverished condition of the people compels them to live as best they can in the parts of buildings that remain standing. It was in these localities that great loss of life occurred.

Minimizing Hurricane Damage.—All buildings in Cuba are fitted with shutters on windows and doors. This is a detail that helps greatly in minimizing hurricane damage, for the shutters protect the window glass and keep the wind out of the interior of the building. Close spacing of dwellings, especially in the older residential areas, is another detail that

minimizes damage from storms, for the buildings protect each other and damage is likely to be confined to accessory parts.

Modern Buildings.—Modern buildings in Havana are rapidly coming into prominence. Like their predecessors, all modern buildings of archi-

FIG. 24.—TYPICAL DESTRUCTION OF OLDER FORMS OF CONSTRUCTION.

These buildings, on Companario St., near Malecon Drive, Havana, are built of native stone. The location is near the ocean front. Failures of this nature were far more frequent in the smaller cities and towns than in Havana, where most of the damage was confined to parapet walls, and accessories such as balconies and shutters.

tectural pretension have their exteriors finished in stucco. The effects obtained are wonderfully pleasing.

Almost without exception the modern buildings in Havana have either a reinforced-concrete structural frame or a concrete-encased steel frame,

the proportion being about 40 per cent of the former and 60 per cent of the latter. In not a single instance did a reinforced-concrete building suffer any structural damage. This is true of the areas outside of Havana as well as those within the city.

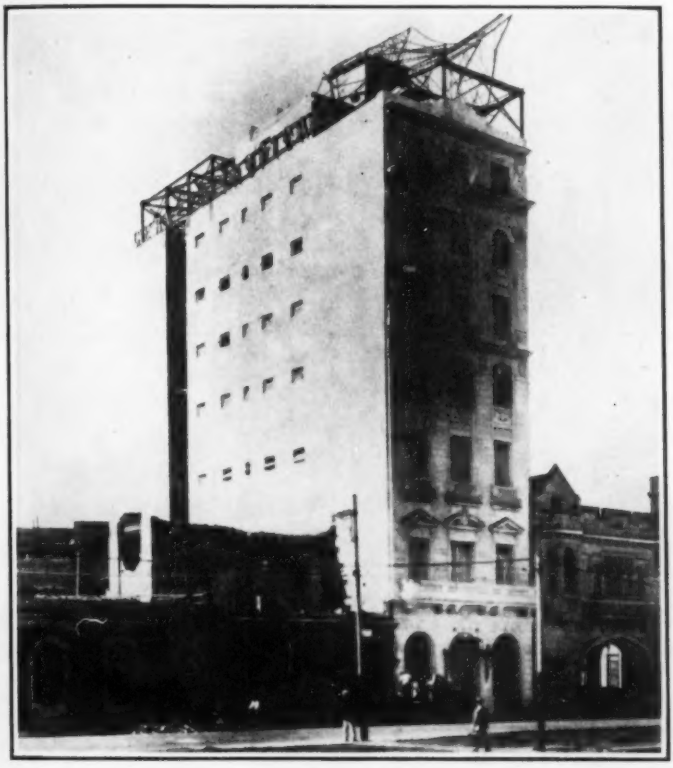

FIG. 25.—ECHEVERRIA APARTMENTS, 16 MARINA ST., HAVANA.

The perfect performance of this seven-story reinforced-concrete apartment building on the ocean front, built in 1925, is characteristic of the performance of reinforced-concrete structures of all kinds in the hurricane that swept Havana on Oct. 20, 1926. Exposed to both sea and storm, it suffered no damage aside from broken windows. The old type stone building on the left of the apartment building was destroyed, and the steel framework on top of the apartment building was hopelessly distorted.

Completed steel frame buildings of the office and residential class likewise suffered no damage. However, a four-story apartment building of the steel frame type, under construction at the corner of 25th St. and the Paseo, was completely destroyed. Steel industrial buildings of the trans-

verse bent type, employed extensively in the construction of sugar mills, registered a number of costly failures. (See Figs. 28, 29 and 30.)

FIG. 26.—CARRERA JUSTIZ APARTMENTS, FACING THE OCEAN FRONT ON MALECON DRIVE, EXTENDING THROUGH TO SAN LAZARO ST., ON THE WEST SIDE OF ANTONIO MARIA LAZCANO ST., HAVANA.

Eight stories high, reinforced-concrete structural frame and floors. This is the first apartment building built in Havana. It was built by Antonio Rodriguez. Although a long, narrow building, exposed on all sides to sea and storm, it suffered no damage whatever in the hurricane of Oct. 20, 1926. This photo was taken on Oct. 29, 1926. Several of the low buildings on each side of it suffered serious damage in loss of parapet walls and rear walls.

Among the modern reinforced-concrete buildings in Havana, the Echeverria Apartment Building, at 16 Marina St. (Fig. 25), deserves special mention. This building, a long, narrow seven-story structure located on

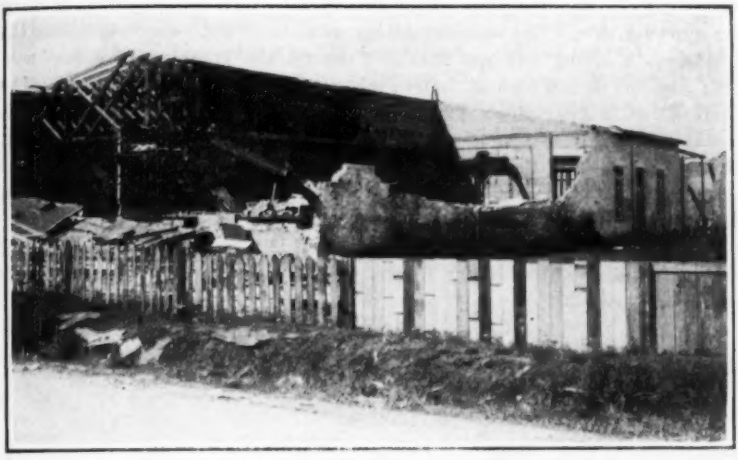

FIG. 27.—TYPICAL FAILURE OF AN ISOLATED BRICK BUILDING.

This is the Regia Ice Company's building at Embil Station, in Regia, a suburb of about 8,000 population, across the harbor from Havana. Cuban hurricane, Oct. 20, 1926.

FIG. 28.—TOLEDO SUGAR MILL, NEAR MARIANAO, STEEL BENTS WITH BRICK CURTAIN WALLS.

Steel frame badly twisted and completely destroyed. Cuban hurricane of Oct. 20, 1926.

the ocean front, was exposed on all sides to the action of both wind and sea. The building was struck first on one side, then on the front, then on the opposite side. The stone buildings next to it were destroyed, but the apartment building was not damaged beyond the breaking of a few windows and the destruction of framework supporting signs on the roof. The Carrera Justiz Apartments, facing the ocean front on the Malecon Drive, is another splendid example of reinforced-concrete construction (see Fig. 26). This structure has the distinction of being the first modern apartment built in Havana. It is also on the ocean front, but it was not damaged in the storm.

Concrete Bearing Walls.—Plain concrete is a common form of bearing wall in buildings of ordinary height. No failures occurred among walls of this material.

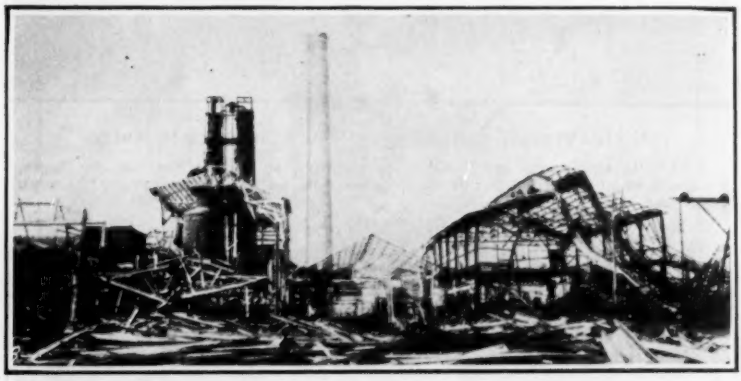

FIG. 29.—SUGAR MILL AT LA JULIA, NEAR HAVANA.

All that is left of this mill is the company's reinforced-concrete chimney. The building, of steel frame and corrugated iron construction, were completely destroyed in the hurricane of Oct. 20, 1926.

Three Types of Buildings that Failed.—Mention has previously been made of the general failure of wooden buildings such as warehouses, dock structures and sheds. Wooden buildings seemed to be too light to offer successful resistance.

Brick walls likewise failed quite extensively. Clay brick available in Havana is of rather poor quality. The many failures of walls and of entire buildings of this material mark it as unsuitable for that locality. Brick panel walls in skeleton buildings were in a number of instances blown out (see Fig. 21). Fig. 27 is a typical example of the destruction of entire buildings of brick.

Another type that suffered extensive failure is the so-called steel mill building, consisting of transverse steel bents, not fireproofed, and curtain walls or wall covering of one of various kinds. This type is employed

quite generally in the construction of sugar mills and other large industrial buildings. Figs. 28, 29 and 30 are typical examples of the destruction of buildings of this type.

Building Regulations in Havana.—Because of the generally good record made in the storm by modern buildings, the logical inference is that the officials in charge of the inspection of structures must have the situation well in hand. This is true, but not in the sense that might be supposed.

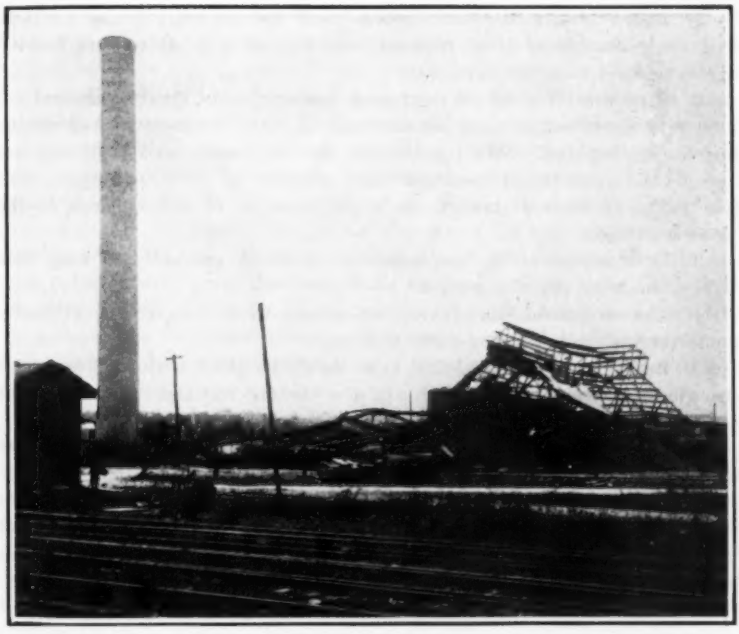

FIG. 30.—REGLA SUGAR REFINERY, AT REGLA, A SUBURB ACROSS THE HARBOR FROM HAVANA.

This was a five-story steel frame building with corrugated iron sides and roof. It was flattened out endwise and completely wrecked in the hurricane of Oct. 20, 1926. The reinforced-concrete chimney, designed and built by the General Concrete Construction Co., of Cuba, is all that remains of the plant. The chimney was not damaged.

The secret lies in the fact that the designer of a building is held directly responsible for its structural safety. This responsibility has been established, not through a clause in the building laws, but through cases actually tried in the civil courts.

Under this system the municipal inspection department has very little field inspection to do. The designer, knowing that he will be held responsible for defective work, sees that honest construction goes into the building. *He* provides the necessary field inspection.

This method of handling this troublesome question has led to the growth of a system peculiar to Havana. Instead of having one individual or firm acting as architect, another as structural engineer, and a third as contractor, these three functions are all centered in one person or one organization. This serves further to place the responsibility for structural safety.

Conclusions.—The conclusions to be drawn from the effects of the Cuban hurricane may be summarized as follows:

1. Under present-day construction costs the concentration of strength in a continuous structural frame is the logical type of construction in regions subject to violent storms.

2. Structural frames of reinforced concrete or of structural steel encased in concrete can be designed and built to resist the most violent storms thus far experienced. Structures which are reasonably well built and are properly designed for an assumed wind pressure of 30 lb. per sq. ft. will have sufficient reserve strength to resist a storm of the violence of the Cuban hurricane.

3. Reinforced-concrete construction, although generally of only fair quality, made an enviable record.

4. The structural steel frame, when encased in concrete, is virtually as effective as the reinforced-concrete frame.

5. Buildings of the steel mill type should be fitted with steel sash and wire glass, to keep out the wind, and the curtain walls should be built of a material that will add to the rigidity of the structure.

6. Clay brick construction proved itself unequal to the demands for tall chimneys in hurricane areas.

7. Stone and clay brick masonry laid in the ordinary way cannot safely be depended upon to resist violent hurricanes unless the individual masonry units are of good quality, and unless a high grade of workmanship and a good quality of mortar can be assured.

8. A fundamental requirement is that the wind must be kept out of the interior of buildings. Windows protected by strong shutters will go far toward accomplishing this purpose.

9. Making the designer of a building responsible for its structural safety has been an important factor in the prevention of unsafe construction in Havana.

10. The exposed situation of the taller modern buildings in Havana will give an excellent opportunity to estimate the outside limit of the force of the hurricane against structures, expressed in pounds per unit of area.

Discussion.—Hurricane Damage in Florida.

J. C. Pearson (*By Letter*).—In view of the damage to concrete block construction wrought by the hurricane of Sept. 18, 1926, in the Miami district in Florida, a tour of inspection was made to determine whether the failures were due to any inherent weakness in concrete block masonry or whether they were to be mainly accounted for by poor quality of materials and cheap construction. The accounts of the storm damage submitted by representatives of associations promoting particular types of building units seem to be unduly enthusiastic over the performance of their particular products, and to lack evidence of much analytical study on the part of the writers. Similarly, the reports of most of the engineers are devoted mainly to the behavior of the larger structures and dismiss the tile and block failures with brief generalizations on poor construction. But there are many points of interest in the masonry failures which seem to bring out at least one important lesson. This lesson and a few additional interesting points can best be illustrated by a few typical views which will now be shown. Mr. Pearson.

By observing the views herewith you will note how rarely any considerable section of overturned block walls or parapets have fallen without breaking up into the individual units. This was noticeable in the great majority of cases examined and seem to furnish convincing evidence that most of the concrete block failures, where not caused by impact or inadequate design against wind pressure, are directly traceable to *weak mortar*. Of the 40 or 50 cases of damaged block construction examined there was only one single instance in which both blocks and mortar appeared to be in substantial accord with the American Concrete Institute specifications. In this case the building was wrecked by impact, or buffeting or undermining by the heavy seas, but large sections of the overturned walls were still intact.

Fig. 1 shows one of the typical cheaply constructed block garages in Miami, examples of which are not by any means confined to the state of Florida. The rear portion of this structure collapsed—what prevented the remainder of the building from doing likewise is one of the unexplained freaks of the storm. The mortar in these walls apparently contained very little, if any, cement, and the weakness of the bond between mortar and block is shown by the diagonal cracks following the joints, which evidently developed when the roof was pushed forward by the collapse in the rear. From such examples as this it is evident that the strength of such walls is governed by the qualities of the mortar rather than by any fault in the strength or design of the concrete blocks.

FIG. 1.—WEAKNESS OF A CEMENT BLOCK WALL LAID IN LIME MORTAR.

FIG. 2.—REMAINS OF A PUBLIC GARAGE AT MIAMI.

Cheapness of construction is indicated by poor quality of concrete in pilasters, lack of tie beams, lightness of wooden roof truss, lack of knee bracing, and poor quality of blocks and mortar, indicated by the character of the debris in the foreground.

Fig. 2 shows the remains of a public garage, the rear part of which collapsed entirely, the front part is being apparently held up by interior partitions. The inferior construction is indicated by the soft honeycombed concrete pilasters without tie beams, the light wooden roof-truss which is considerably distorted, and by the soft tile and mortar. The poor quality of blocks and mortar is here only one of the contributing factors in the failure as the absence of bracing against wind pressure would probably have permitted the collapse to occur even if the wall panels had been of good quality. This building fell toward the south which indicates that it collapsed during the first and less severe half of the storm. Another weak structural detail is shown by the lack of a tie between the wall panel and the concrete pilaster. Frequently these walls are built without securely grouting the space between panel and pilaster. Better construction is obtained by pouring the pilasters after the wall panels are built; the latter serving as part of the pilaster form. Finally the pouring of a reinforced-concrete beam or lintel to tie both wall sections and pilasters together would do much to insure the wall's acting as a unit. Building in this manner with adequate pilaster design, not only in regard to strength, but also in regard to footings, would seem to be cheap insurance against destruction by lateral forces of every reasonable degree.

Incidentally, the building unit used in this case was concrete tile, a smaller unit than the 8 x 8 x 16-in. concrete block. This was one of the very few instances of concrete tile construction in Miami. In general the larger blocks are almost universally used there. The tile is generally conceded to be the largest one-hand unit that the mason will so handle, and many of the tiles are so designed as to facilitate this handling. Furthermore, concrete tiles which meet the Standard American Concrete Institute Specifications for quality are of necessity composed of stronger concrete than the thicker walled block which meet the same specifications. In several respects the concrete tile is the preferred masonry unit and it could well find a larger place in the Miami territory.

Fig. 3 shows another garage which suffered heavy damage in collapse of roof and parapet walls. The concrete framing of this structure follows the better plan just mentioned, but the poor quality of the concrete is plainly indicated in this view. Exceedingly poor quality of both blocks and mortar is perhaps mainly responsible for the failure in this case as is indicated by the quantity of broken blocks scattered about the place. At the time this garage was visited the parapets were being rebuilt and all the unbroken blocks were being salvaged and used again. There was no difficulty in removing the original mortar which should have been in this case much harder than the blocks themselves. In checking up the mason on this job, it was found that the mortar mixture used in reconstruction apparently consisted of about 2 parts of lime to 1 of cement with as much sand as would make a good working mortar. A building inspector, present at the time, warned the mason that he was required to use at least two parts of cement to one of lime. The Miami building code, however, requires that

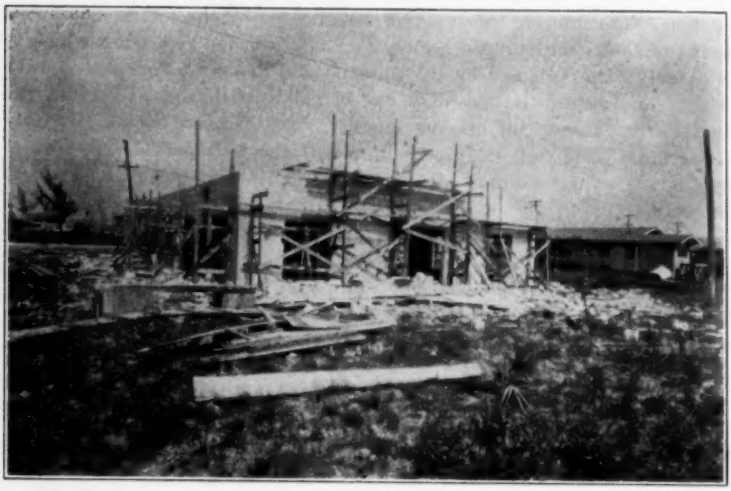

FIG. 3.—PARAPET AND ROOF FAILURE DUE PRIMARILY TO POOR QUALITY OF BLOCKS AND MORTAR.

Note the quantity of unsalvagable material remaining on the premises.

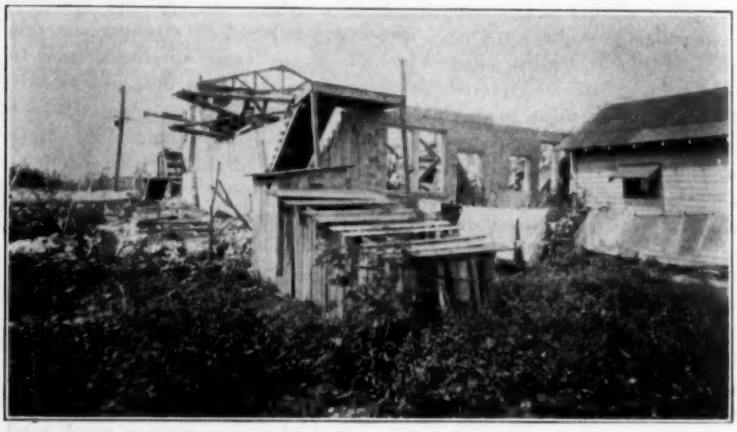

FIG. 4.—COLLAPSE DUE TO AN OPEN SECOND STORY HALL CONSTRUCTED WITHOUT PROVISION FOR WITHSTANDING LATERAL PRESSURE.

all concrete blocks shall be laid in 1 : 3 cement mortar, in which lime may be substituted for cement up to 15 per cent by volume of the latter. This is nearly the equivalent of the A. C. I. specification requirement, but it does permit the use of a slightly leaner mortar. Nevertheless, no evidence was found that such mortar is used except under the most rigidly enforced specifications.

While emphasizing continually the strength of the mortar it is not the intention to minimize the effect of the poor quality of the block. The strength of the block is equally as important as the strength of the mortar; the practical objection to the weak block is that it is highly absorptive and for this reason it quickly robs the mortar bed of its moisture and consequently of much of its strength and bond. For this reason good quality in the block is essential. The Miami building code now requires that concrete blocks shall stand a compressive test of 800 lb. per sq. in. of gross area and this is quite rigidly enforced. On a number of reconstruction jobs it was very evident that the new blocks were of a much better quality than those used originally.

Fig. 4 shows what is left of a building in the northern part of the city. The lower story was a garage, and the upper story a hall used for meetings of some labor organization. The blocks and mortar in this building were inadequate but better than in many structures which suffered a like fate. Inside the building and in the left foreground you will note that the blocks are pretty well separated, but up along the south wall is a considerable section which held together—an evidence of at least some good mortar with good bond. The poured concrete frame of the lower story also approaches good design but failure was due to the open interior construction of the second story hall. There was little lateral resistance there and collapse occurred in the early part of the storm. You will note that one section of the second story remained in place, due probably to the support of the interior partitions closing off small rooms in this portion. This case like several others shown here illustrates the weakness of box construction without interior bracing or support. On the corner nearest the camera is an interesting example of good bond between pilaster and wall panel. These pilasters were poured after the wall panels were built up. The impact of collapse on the south wall tending to overturn it produced a vertical crack in the east wall extending through the blocks instead of through the joint between pilaster and panel. In many buildings in which there was not a secure tie between frame and panel, even though the joint were grouted, the result was bodily displacement of the panel, inward or outward.

In Fig. 5 are seen the remains of a moving picture studio on South Miami Avenue. The frame of this building was made of very excellent concrete and the curtain walls of interlocking terra cotta tile were laid in good mortar. Numerous large sections of tile wall were unbroken. One such section is seen wedged between the wooden poles at one corner of the building. Failure was due to lack of lateral resistance, attributable partly

FIG. 5.—FAILURE OF A MOVIE STUDIO, BUILT WITH GOOD MATERIALS AND WORKMANSHIP, BUT INADEQUATELY DESIGNED AGAINST PRESSURE.

FIG. 6.—TYPES OF FENCE-WALL FAILURE AT CORAL GABLES.

to lack of pilaster footings and partly to lack of internal bracing. This is one of many examples of large open interior construction which failed, not from poor quality of workmanship or materials, but through inability to withstand high wind pressure.

Fig. 6 shows some of the damage to the Miami Biltmore property at Coral Gables. The hotel itself was not damaged structurally but the large unfinished wall surrounding the parking space and the walls lining the entrance driveway to the service building as shown here were practically destroyed. The wall on the right was overturned because of inadequate footing. The coral or lime rock formation which in the Miami district extends practically to the surface of the ground tends to discourage the construction of footings for light and medium structures. The bearing power is entirely sufficient for ordinary loads, but structures so built have comparatively little resistance to overturning moments. The wall on the left has in effect a wide concrete footing. The anchorage provided by this footing prevented the overturning of the wall as a whole, but a portion of the upper part collapsed. The wall posts were not poured integrally with the base and the post reinforcement did not extend into the base more than 10 or 12 in., thus providing inadequate support. In spite of this feeble resistance to side thrust only a part of the wall superstructure succumbed to the first part of the storm, the three posts in the left foreground showing that they fell after the wind had changed to the south.

In Fig. 7 we have an interesting example of what good mortar and stucco can do. This is one of the residential properties on the bay shore. The enclosing wall was overturned and broken by the force of the pounding water and floating debris. Of the two wing walls at the entrance gate one remained intact; the other was overturned but did not break. Each wing wall is practically a quarter cylinder of 4-in. brick masonry laid with good mortar and covered with a very hard coating of cement plaster on each side. The strength of the mortar and plaster were sufficient to hold this 4-in. layer of brick together like a solid piece of concrete, even though the impact forces to which it was subjected must have caused stresses much greater than the transverse stresses set up by falling in the position shown.

Fig. 8 is a view of the well-known Methodist Church at Hollywood. The construction comprised panel walls set in arched concrete frames. These panel walls were of interlocking terra cotta tile laid in absolutely first-class mortar. In fact it was the best mortar encountered on the whole tour of inspection. Possibly this fact contributed somewhat to the early collapse of the main part of the structure in that the panel walls may have held until the body of the church went over. There was no opportunity to make a study of this failure, but on brief inspection it seemed that the reinforcing of the frame was very light and that this with the lack of interior bracing permitted the wind to do its work in the early part of the storm. Like most of the other cases of open interior construction, this building failed before the maximum force of the hurricane was brought to bear upon it.

FIG. 7.—ENTRANCE WING WALLS OF BRICK MASONRY AND CEMENT STUCCO SHOW REMARKABLE STRENGTH IN RESISTING THE IMPACTS OF HEAVY SEAS AND FLOATING WRECKAGE ON THE BAY SHORE.

FIG. 8.—THE METHODIST CHURCH AT HOLLYWOOD.

Fig. 9 is an example of what would at first glance be called freak storm damage; three adjacent small houses; the center one seriously damaged; the outer ones untouched except for the roof covering. But in this case something is known about the construction of the center house and the one at the right. The latter was built of good frame construction covered with metal lath and stucco. The damaged house shows definite evidence of cheapness in the quality of blocks and mortar, in the unreinforced lintel over the window at the right, in the missing roof anchors and in the light built up wooden beam which supported the front of the porch roof. This beam was covered with scraps of wood lath and a coat of stucco. The extensive damage presumably occurred during the first part of the storm, whereas not even the outside fireplace chimney of the next house was disturbed.

Fig. 10 is an example of extreme damage to stucco on wood lath. Evidences of stucco failure on wood lath were rather prevalent but failure of stucco on metal lath was not found at all. The explanation may be that the wood lath swelled from the long continued deluge, resulting in the cracking off of some of the plaster keys, and the wind did the rest. This particular type of failure does not come under the head of structural damage, but it does have a decided bearing on the proper use of portland cement stucco. The American Concrete Institute has wisely refrained from approving the use of wood lath in its stucco specifications.

In summarizing observations in the storm area the general impression is that the concrete block damage may be largely accounted for by two conditions: first, the general lack of standard quality in the blocks that failed and the general disregard of building code requirements and standard recommended practice in laying the blocks, and second, the lack of lateral bracing in the ordinary buildings. The latter condition is one of design and is more or less independent of the quality of the blocks and mortar, although not entirely so. Builders in the Miami district can hardly be blamed for not having built with thought of what a hurricane like that of Sept. 18 might do, but the experience has served to uncover much of the inherent weakness in cheap construction and shows quite clearly what measures of protection may be taken against a repetition of such a disaster.

The points which should be emphasized are as follows:

1. The strength and unity of masonry walls are dependent in large part on the strength of the mortar. This is the first and most important item in providing low cost insurance against storm damage.

2. Blocks of standard quality will insure that good mortar properly placed will develop its full potential strength and bond. The present Miami building code requirements, if enforced properly, amply take care of these first two items.

3. Concrete tile of standard quality have some advantages over the larger concrete block in making for more rigid construction. There is a large field for these units in southeastern Florida.

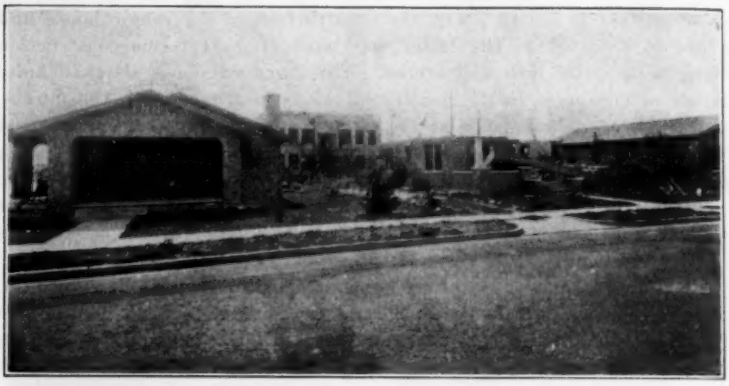

FIG. 9.—ILLUSTRATING THE DIFFERENCE IN QUALITY OF CONSTRUCTION IN SMALL HOUSES.

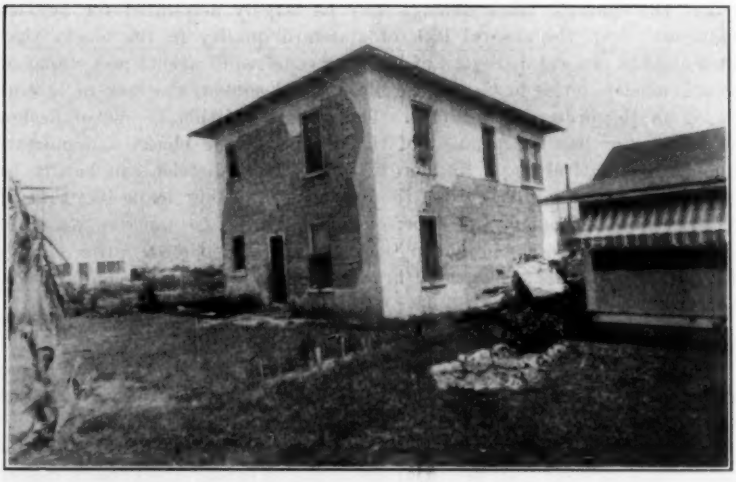

FIG. 10.—AN EXAMPLE OF EXTREME DAMAGE TO STUCCO ON WOOD LATH.

4. Design of ordinary concrete block buildings calls for particular attention to means of providing necessary support against wind pressure. Positive ties should be provided between the walls and the roof, the floors and the partitions. The proposed new requirement that all masonry parapet walls shall be capped with a continuous reinforced-concrete coping is good.

5. In buildings of the public garage type, small halls and other small buildings of open interior construction, continuous reinforced-concrete framing built up with the walls is to be recommended. In this type of construction the contribution to lateral strength afforded by extended pilaster footings should not fail of consideration.

6. The generally excellent performance of concrete block masonry in the city of Coral Gables, and in fact in the great majority of buildings in the city of Miami, notwithstanding the adverse effect of numerous failures of inadequately designed and poorly built structures, should give sufficient assurance to the unbiased that there is nothing intrinsically wrong or weak in this type of construction.

The comparatively low price of concrete blocks and the still lower price of sub-standard concrete blocks has encouraged their use in speculative building, and the local industry in Florida is suffering from the reaction. In the last analysis this unfavorable impression is simply the result of abuse to which all unrestricted concrete products are peculiarly liable. Given the protection which our building codes and our standard specifications provide, and which the industry must largely demand for itself by everlastingly insisting upon the maintenance and observance of these standards, concrete masonry should prove dependable, economical and entirely satisfactory under very severe storm conditions.

DISCUSSION.

Mr. Stineman.

MR. STINEMAN.—Some one has said that wherever he found a concrete block building he found a failure. A number of similar statements have been made and published and have led to a wholly erroneous impression of the performance of concrete block in the Florida hurricane. I went through Coral Gables twice—once under the leadership of the building inspector of Miami. He made a statement to the effect that "Here is an example of what you get when you start right and keep going right."

There is nothing superhuman about Coral Gables. They have just an ordinarily good building inspection department, and they had just ordinarily good construction. Out of 3,500 buildings in that city, 93 per cent are concrete masonry, and they had only about 3 per cent of failures.

Last April I was told at the Twelfth Annual Building Officials Conference by L. F. Fletcher, building inspector of Orlando and president of the newly-created Florida Building Officials' conference, that it was not a question of what is going to happen if a major hurricane occurs, but simply a question of when it is coming. Georgia red clay tile, he said, was being shipped into Florida and used there, although its quality was so inferior that cities where building inspection departments are functioning will not permit it to be used. He also said concrete block makers were setting up concrete block establishments in tents or sheds, using fine beach sand full of palmetto roots, and making block so poor in quality that it barely hangs together until it is placed in the wall.

When I went to Florida after the hurricane I expected to find general antagonism toward concrete masonry, on the part of building inspectors, but instead of that I found none. They know very well what took place. They know that this block of poor quality made during the boom and used by the jerry-builder was so highly absorptive that, whether the mortar was good or poor, the block promptly drew the greater part of the water out of it. Consequently no bond was formed between the mortar and the block. These building inspectors think they can prevent that kind of construction in the future, for the boom is over now. They are going to require tests and the anchoring of walls to foundations, the anchoring of floors and roofs to walls, and the use of pilasters and other means of getting lateral strength. They think, and think with good reason, that they can now prevent the sort of construction they had in 1925 and in the early part of 1926, and there is no reason for anyone to get excited about concrete masonry in Florida. The reason so much of it failed was because there was so much of it used by the jerry-builder. You must distinguish between what was built before the boom and what was built during the boom. Here was a material they could pick up anywhere. There is nothing so plentiful in Florida as sand. It is of very poor quality, it is true, but there is a great deal of it. If the dishonest block maker could get a little cement to go with it, he could transform it into block anywhere.

Qualifications of Different Kinds of Natural Stone for Concrete Aggregate.*

By G. F. Loughlin.†

Introduction.

Concrete has not always given complete satisfaction and its imperfections have been attributed to different causes. Accumulation of evidence has given the impression that one of the causes that has not been fully appreciated is the poor quality of aggregate that has too often been used in concrete. The usual physical tests are adequate for the determination of some of the qualities of aggregate, but do not indicate certain other qualities, particularly resistance to prolonged weathering. It is the object of this paper, therefore, to supplement the usual test data by calling attention to the fundamental physical and chemical properties of natural stone and to the consequent behavior of different kinds of stone when used as concrete aggregate. The writer's viewpoint is that of the geologist interested in the correct use of stone but inexperienced in the mixing and handling of concrete. A thorough appreciation of the use of natural stone in concrete obviously calls for close co-operation of the geologist with construction engineers.

The fundamental properties of rocks including gravel and sand depend upon the chemical composition and structure of the rock-forming minerals and the manner in which they are put together—in other words upon mineral composition and texture. Only natural rocks are considered in this paper.

Mineral Composition and Texture of Rocks.

The formation of rocks during the history of the earth has taken place in a series of partial to complete cycles. The earliest formed were igneous rocks formed from molten magma. These, by decomposition and erosion, gave rise to sedimentary rocks. Deep burial of both igneous and sedimentary rocks beneath thick accumulations of sediment subjected them to new igneous intrusions and deep seated compression, which changed their form and in part their mineral composition and converted them into metamorphic rocks. Upheaval and renewed erosion again brought these metamorphic and new igneous rocks to the surface, and a new series of sedimentary rocks was derived from them, marking a new cycle in the earth's history. At some times sedimentary rocks after only shallow burial and partial consolidation were elevated, eroded and redeposited without passing through a complete cycle, and the new rocks thus formed therefore contained fragments of the older strata. With this brief state-

* Published by permission of the Director, U. S. Geological Survey.
† Geologist, U. S. Geological Survey, Washington.

ment in mind, it can be appreciated that rocks as we find them today present a great variety of contrasts in composition, alteration, decomposition, texture, and degree of consolidation. These variations are most conveniently considered under the three main classes of rocks—igneous, sedimentary, and metamorphic. It is noteworthy that in recent correspondence on the subject of unsound concrete aggregate engineers have referred only to sedimentary rocks, but poor concrete due to altered igneous rocks and doubtless to certain metamorphic rocks may be cited. Adequate consideration must therefore be given to each of these three main groups of rocks.

IGNEOUS ROCKS.

The principal igneous rocks classified according to mineral composition and texture are named in Table I. This table is adapted from "Rocks and Rock Minerals,"[1] one of the best elementary text books on petrology. Names of the most abundant and most commonly used rocks are underlined. Many other names of igneous rocks may be found in the literature on petrology, but from the standpoint of the construction engineer they are to be considered varieties of those in the table.

The names within a vertical column represent rocks of similar mineral and chemical composition; for example granite and rhyolite are derived from the same kind of material, consist of the same essential minerals, and differ only in grain. Granite crystallized at considerable depth under conditions that permitted single crystals to grow large enough to be readily seen by the unaided eye, whereas rhyolite consolidated near or at the surface so rapidly that most of its mineral grains are of microscopic size. Where lava of the same composition consolidated so quickly that the greater part of it could not crystallize at all, it formed a volcanic glass or obsidian. Explosive eruptions of the same lava forms deposits of fragments and dust which became cemented together to some degree, forming cones and necks or pipes of large and small fragments (agglomerate) and layers of prevailingly small fragments and dust (tuff).

The granular rocks can be readily distinguished by one acquainted with the common rock-forming minerals, but those mainly or completely of microgranular texture and not very different in mineral composition may not be distinguishable without microscopic examination. In field usage, therefore, if it is impossible to identify a rock as rhyolite or trachyte, it is given the more general name of felsite.

Different names within a single box in the table imply for the most part the prevalence of different kinds of feldspar; thus in granite and syenite the potash or potash-soda feldspar equals or exceeds the soda-lime feldspar. In monzonite both feldspars are present in about equal amounts, whereas in diorite the lime-soda feldspar greatly predominates. The difference between gabbro and diabase is wholly textural and structural. Gabbro is relatively coarse grained and forms large irregular masses, whereas

[1] Pirsson, L. V., Rocks and rock minerals (revised by A. Knopf in 1926). John Wiley and Sons, New York City.

diabase is relatively fine grained and forms either small to large dikes intruded along fissures, sills intruded along bedding planes of sedimentary rocks, or thick flows on the surface. Diabase is furthermore characterized by an "ophitic" texture produced by the haphazard arrangement of light colored rod or lath-shaped feldspars in a black mass of pyroxene grains. Where this texture is not detected and the black mineral cannot be identified, fine grained rocks are designated by the general term dolerite.

TABLE I—ELEMENTARY CLASSIFICATION OF IGNEOUS ROCKS

		Feldspathic (acidic)		Intermediate	Ferromagnesian (basic)	
Mineral Composition	Essential minerals	Feldspar and Quartz	Feldspar alone	Feldspar and black silicates (hornblende, mica, pyroxene)	Black silicates (pyroxene, hornblende, mica) and feldspar with or without olivene	Black silicates alone or with olivene
	Major accessory minerals	Mica, hornblende, (pyroxene)	Hornblende, pyroxene, mica		Magnetite Ilmenite	Magnetite Ilmenite
Texture and Origin	Visibly granular (deep seated intrusive)	*Granite Granodiorite, Quartz monzonite*	Syenite (nephelite syenite (Anorthosite)	Monzonite *Diorite*	*Gabbro Diabase*	Pyroxenite Amphibolite or Hornblendite Peridotite
				Dolerite		
	Dense or microgranular (surface flows and intrusive near earth's surface)	*Rhyolite Dacite*	Trachyte (Phonolite)	(Latite) *Andesite*	*Basalt*	
		Felsite			*Basalt*	
	Glassy (surface flows)	Obsidian (Pumice)				
	Fragmental explosive volcanic eruptions	Volcanic breccia and tuff (volcanic ash)				
Subordinate Textures and Structures	Porphyritic	Relatively large crystals in fine grained, dense, or glassy ground-mass—e. g. granite porphyry.				
	Flow structure	Wavy, parallel layering or banding in lava flows and along margins of small dikes				
	Visicular	Small holes formed by expansion of steam in consolidating lava.				
	Amygdaloidal	Visicles filled with alteration products of lavas containing them, or with introduced metallic minerals.				

Diabase and basalt are the original "trap rocks," but this name has been applied by stone producers to so many different kinds of rocks, including dark colored sandstones, impure limestones, and dark fine grained metamorphic rocks, as well as several kinds of igneous rocks, that its value even in non-technical descriptions is largely destroyed.

Another indefinite term that has been considerably used, especially in western mining regions, is porphyry. It has been made to include many kinds of igneous rocks, especially those of fine grained or dense texture, but

its strict meaning implies an igneous rock of any composition consisting of relatively large crystals of one or more minerals imbedded in a fine grained, dense, or glassy groundmass.

A glance at this table shows that igneous rocks are distinguished by certain differences in texture, structure, and essential mineral composition. It is therefore a consideration of the textures, structures, and minerals rather than of single kinds of rocks that is of most present interest.

Textures and structures.—As granular texture in igneous rocks is produced by the welding together of minerals under great pressure while they are crystallizing, there need be no concern over the cohesive strength or permeability of granular igneous rocks. This statement is supported by the many records of high crushing strengths and low ratios of absorption. The same statement applies to most of the dense rocks. The glassy rocks also are strong and impervious, but in some of them as well as in some of the dense rocks flow structure has developed partings or planes of easy separation along which water may percolate and induce weathering. The vesicular igneous rocks are obviously not so strong, and are likely to permit slow permeation by weathering agencies, but have the special advantage of furnishing an aggregate of very low specific gravity and strong enough for most practical purposes. In fragmental volcanic rocks the larger fragments represent any or all of the kinds already considered and when crushed are equally satisfactory provided they are unweathered; but their interstices subject them to extensive permeation and weathering, and the crushed product may contain enough poor material to render the whole inferior. Some agglomerates and breccias are quite satisfactory, others are not, and if their use is considered their quality should be determined by microscopic examination and physical and chemical tests. Tuffs because of their porosity and fineness of grain are more subject to weathering than the other igneous rocks. Fragmental volcanic rocks to which these remarks apply are restricted to the Western States. Those in these Eastern States are geologically much older and have been subjected to deepseated alteration that has converted them into impervious rocks.

ORIGINAL MINERALS.

Minerals free from alteration.—Most of the original rock-forming materials can be dismissed from our minds as quite durable from the engineer's standpoint, unless they are considerably altered. If we remember that some of our oldest granite buildings and monuments have been standing more than 100 years without showing appreciable effects of weathering, it is obvious that the essential and major accessory minerals in these granites are extremely resistant to weathering under the conditions of use. The same granites used as aggregate in dense concrete are protected from the weather unless exposed at the surface of the concrete. At the surface they may be subjected to attack by water or by dilute solutions that seep through the concrete or are drawn up from the ground by capillary action. Any alkali or other salts deposited by these waters

in the minute surface cracks of the granite fragments may wedge off minute mineral particles and may cause some chemical reaction, especially if the granite is appreciably altered; but if the minerals in the granite are not excessively cracked and are not seriously altered serious damage is not to be expected. It is conceivable that in highly permeable concrete these chemical reactions may take place to an appreciable degree within the concrete, but the fault here is due rather to poor work than to poor aggregate.

The foregoing paragraph applies also to the other visibly granular rocks listed in Table 1 which are commercially known as granites if light

TABLE 2—COMMON CONSTITUENT MINERALS OF IGNEOUS ROCKS

Original	Alteration Products	
	Deep Seated	At or Close to Surface
Essential:		
Quartz		
Feldspar:		
Orthoclase	Mica (sericite).	Clay minerals, alkali sulphate and carbonate.
Plagioclase	*Mica* (sericite), epidote, calcite, gypsum, zeolites, scapolite.	Clay minerals, *gypsum*, alkali sulphate and carbonate.
Pyroxene	Fibrous hornblende, *chlorite*, epidote (serpentine), *calcite* (pyrite), gypsum.	Earthy iron oxide, clay minerals, (*gypsum*).
Hornblende	*Chlorite*, epidote, *calcite*, *pyrite*.	
Olivene	Serpentine (and tale), fibrous hornblende, carbonates.	
Major accessory:		
Mica:		
Muscovite (white)		(Clay minerals).
Biotite (black)	*Chlorite*, sericite, *carbonates*.	Earthy iron oxide and clay minerals.
(Feldspathoids):		
Nephelite	Zeolites (analcite, *laumontite*, *natrolite*, stilbrite, etc.).	Clay minerals.
Sodalite	Sericite.	
Noselite		
Hauynite		
Cancrinite		
Leucite		
(Analcite)	Zeolites.	
Minor accessory:		
Magnetite	(*Pyrite*).	
Ilmenite		
Hematite	(*Pyrite*).	
Apatite		
Zircon		
Pyrite and marcasite		Earthy iron oxide and sulphuric acid.

colored, and "black granites" if dark colored. Plagioclase and the black silicate minerals are more subject to weathering than are orthoclase and quartz, but the difference in rate of weathering of the unaltered minerals is inappreciable unless measured by centuries.

Of the unaltered original minerals the white and black micas, muscovite and biotite, are the only ones that need special attention. Muscovite is one of the most resistant minerals to strictly chemical weathering, and if present in isolated distinct flakes rather thinly scattered through the granite, it is as resistant to weathering as quartz and orthoclase. Even where it forms comparatively thick tabletlike crystals it remains unaf-

fected for a long time unless bruised by the hammering or crushing of the rock. Then its thin, perfectly smooth, cleavage flakes become spread enough to admit and retain rain and seepage water, which increases the spreading, especially in freezing water, until the swelled crystal finally drops from the stone's surface. This process also takes place in biotite, which is also more subject than muscovite to chemical weathering; but thin scattered flakes of unaltered biotite, such as are present in most

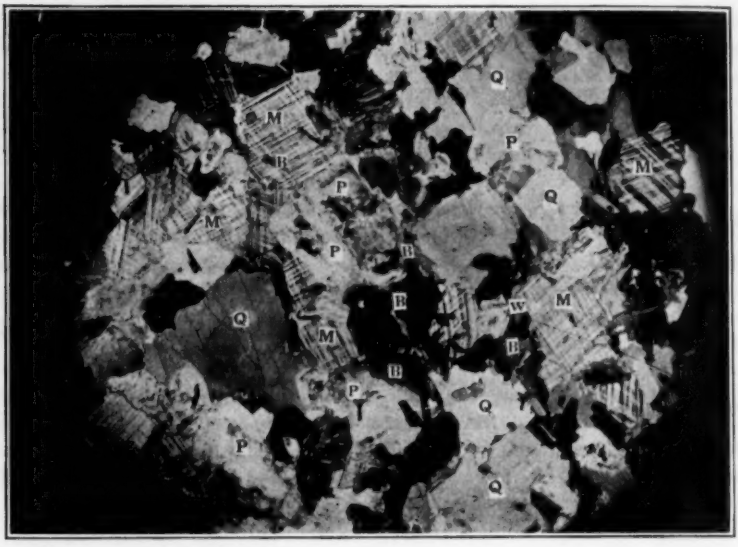

FIG. 1.—PHOTOMICROGRAPH OF UNALTERED GRANITE, SHOWING INTERLOCKING CRYSTAL GRAINS OF QUARTZ (Q), POTASH FELDSPAR OR MICROCLINE (M) WITH CHARACTERISTIC "GRIDIRON" TWINNING; SODA-LIME FELDSPAR OR PLAGIOCLASE (P), BLACK MICA OR BIOTITE (B), AND A LITTLE WHITE MICA (W).

All are free from alteration except the plagioclase which is slightly clouded by a little finely divided white mica (sericite) and epidote. The black mica crystals, though slightly more clustered and abundant than in many granites, are too fresh and too isolated to lower the granite's resistance to natural weathering. The black grains are mainly of quartz and mica that happen to be so situated that they appear black when viewed between crossed nicol prisms of the polarizing microscope. Magnification 20 diameters.

commercial granites (Figure 1), are sufficiently durable for all practical purposes.

Deep-seated alteration.—Igneous rocks absolutely free from any kind of alteration are very unusual, for deep seated agencies of alteration become active as soon as a rock has crystallized and may be continually or intermittently active until erosion has brought the rock into the zone of weath-

ering. These agencies, although spreading throughout large masses of rock and altering them to a slight and insignificant degree, are largely localized. Changes made by them in igneous rocks at extremely high temperatures result in a few very durable minerals, such as tourmaline, topaz, and muscovite, which need no consideration here. At less extreme temperatures feldspars, particularly the soda-lime varieties are commonly converted, wholly or in part, into microscopic mixtures of mica (sericite),

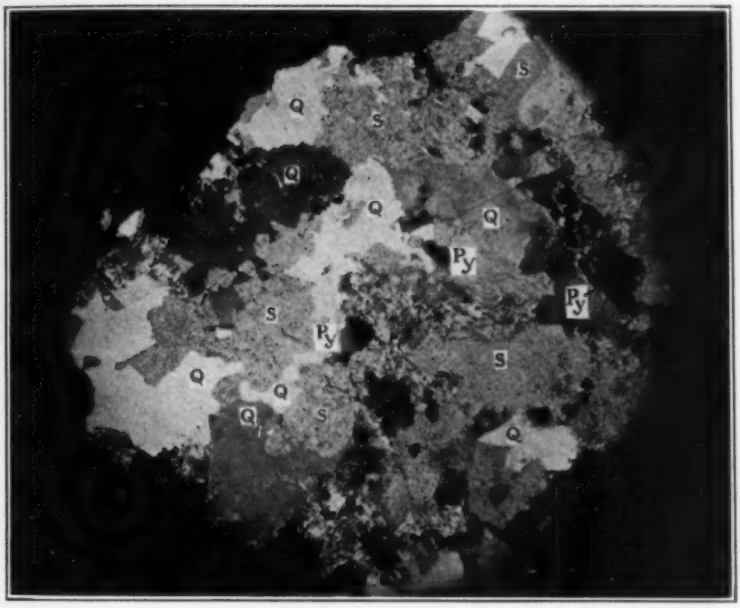

FIG. 2.—PHOTOMICROGRAPH OF GRANITE THOROUGHLY ALTERED BY DEEP-SEATED PROCESSES.

Quartz (Q) is unaffected, but all the feldspars are converted into fine grained feltlike masses of white mica (sericite), which inclose minute particles (not recognizable in the picture) of epidote and quartz. The sericite in turn may be somewhat altered to clay minerals which appear identical with it in the photograph. The jet black grains are pyrite (py) which has been formed from iron originally present in black mica and magnetite, and from sulphur introduced by hot solutions which caused the alteration. Black mica has completely disappeared and its place is taken by clusters of pyrite and relatively coarse grained sericite and chlorite, both of which appear light gray to white in the photograph. Magnified 20 diameters.

epidote, and calcite (Fig. 2), and under some conditions are altered into zeolites; pyroxene and hornblende are altered into mixtures of chlorite, epidote, and calcite; olivene into fibrous hornblende, serpentine, and calcite or some magnesian carbonate; black mica into chlorite, sericite, and perhaps carbonate. These and similar changes are shown in Table 2. They are not as simple as this brief statement would indicate; for example,

constituents from different original minerals may find their way into one alteration mineral, and certain constituents, particularly water, carbon dioxide and sulphur are introduced by the hot solutions or agencies of alteration that permeate the rock. Water forms an essential part of sericite, epidote, chlorite, and the zeolites; carbon dioxide combines with lime and under some conditions magnesium and iron, to form carbonates, and sulphur unites with iron to form pyrite.

The details of the process, however, are of little interest at present compared with the durability of the resulting new minerals. Sericite is

FIG. 3.—PHOTOMICROGRAPH OF FRESH DIABASE.

The white lath-shaped crystals are soda-lime feldspar (plagioclase), the gray irregular grains are pyroxene (augite), and the jet black grains are magnetite or ilmenite. All the minerals are free from appreciable alteration. Magnified 20 diameters.

a variety of white mica, but is so finely divided that it is less resitant to weathering than are the comparatively large crystals and flakes of muscovite. Epidote, a rather light green, very hard mineral, is very resistant to weathering if alone, but is so likely to be intimately mixed with less durable minerals that its green color may be regarded as a sign for caution in the selection of stone. This remark applied to building and monumental stone where small superficial details on finished material may be affected rather than to crushed stone, but is not wholly to be ignored.

Calcite, especially if finely divided, is dissolved by rain water, and if thickly scattered through a rock, considerably increases its tendency tc

weather. Fig. 3 illustrates a fresh diabase which forms the toughest kind of crushed stone and Fig. 4 illustrates an altered diabase in which the original black minerals have been completely replaced by calcite. The altered rock is much softer and less tough than the fresh rock, but will make a firm bond with cement and will prove durable if well imbedded in concrete that is not subjected to frequent seepage.

Members of the zeolite group vary in durability. Probably the least durable is laumonite, which decomposes rapidly on exposure to air, as it did in anorthosite used as fine aggregate in Los Angeles.[1] Others that

FIG. 4.—PHOTOMICROGRAPH OF ALTERED DIABASE, COARSER GRAINED THAN THAT SHOWN IN FIG. 3.

The white lath-shaped feldspars remain free from alteration, but the pyroxene is completely replaced by (gray) fine grained masses of calcite. Magnified 20 diameters.

commony occur in finely fibrous aggregates tend to absorb water and disintegrate, if not decompose. Fibrous hornblende may have a similar tendency, though to a much less degree if pure. Chlorite, a micaceous mineral is similar to black mica, is durable under ordinary conditions of weathering, unless in fibrous or platy aggregates that are porous enough to absorb water and swell (compare Figs. 5 and 6 of fresh and altered basalt). Serpentine also is durable from a practical standpoint if pure, but may

[1] Pearson, J. C., and Loughlin, G. F. Interesting case of dangerous concrete: *Proc.* Amer. Concrete Institute, vol. 19, p. 142, 1923. *Concrete*, vol. 22, p. 144, 1923.

contain so much finely divided carbonate as to disintegrate with comparative rapidity in buildings.

Pyrite is of critical importance. Clean, unaltered crystals of true pyrite have been exposed under ordinary atmospheric conditions for as much as a century without any signs of decomposition, but where attacked by oxidizing waters underground it is one of the first minerals to decompose and to aid in the decomposition of others. Marcasite, different in crystal form, but identical in chemical composition with pyrite, is very

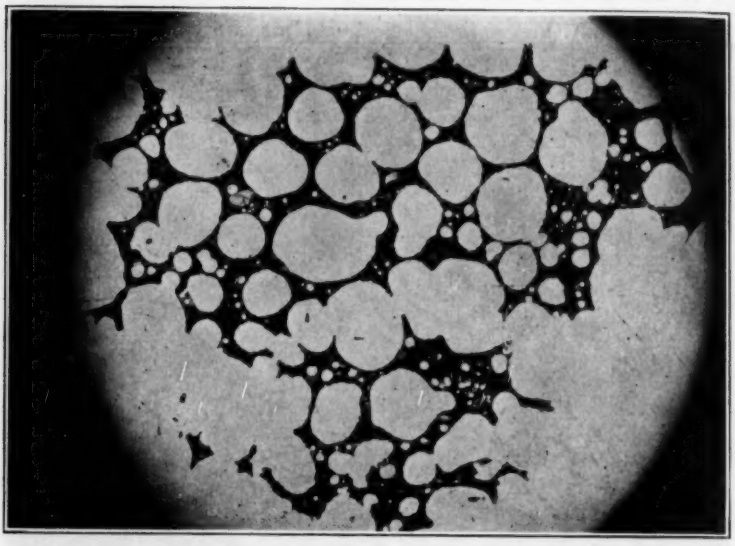

FIG. 5.—FRESH VESICULAR BASALT, COMPOSED OF FINE WHITE LATH-SHAPED CRYSTALS OF SODA-LIME FELDSPAR AND GRAY CRYSTALS OF PYROXENE (AUGITE) IN A GROUNDMASS OF BLACK GLASS OR "NATURAL SLAG."

The large round holes were formed by the expansion of steam liberated when the lava was cooling. These holes are so numerous and so air-tight that a specimen of this lava will float on water for 24 hours or more. This feature together with the fresh condition of the component minerals adapt the rock for use in concrete where light weight is desired. Samples of concrete containing this stone had a crushing strength of 5,000 lb. per sq. in. after 90 days at the Bureau of Standards. This equaled the strength of the stone itself. Magnified 20 diameters.

unstable when exposed to the air, and some specimens of it even decompose in a dry indoor atmosphere. Its influence under natural conditions is similar to that of pyrite but much more rapid.

This influence of pyrite and marcasite upon other minerals is a very important item in the alteration of rocks at or close to the earth's surface, but before that subject is discussed, some remarks are in order regarding the use as concrete aggregate of rocks that have been appreciably

affected by the deep-seated alteration. All the minerals listed in the second column of Table 2 render the rocks that contain them in considerable quantity softer than they were originally. These altered rocks may or may not make any more screenings or dust during crushing than the unaltered rocks, but the fine material will have an appreciable tendency to produce visible weathering effects if present close to the surface of the concrete, and if it contains a large proportion of micaceous material will

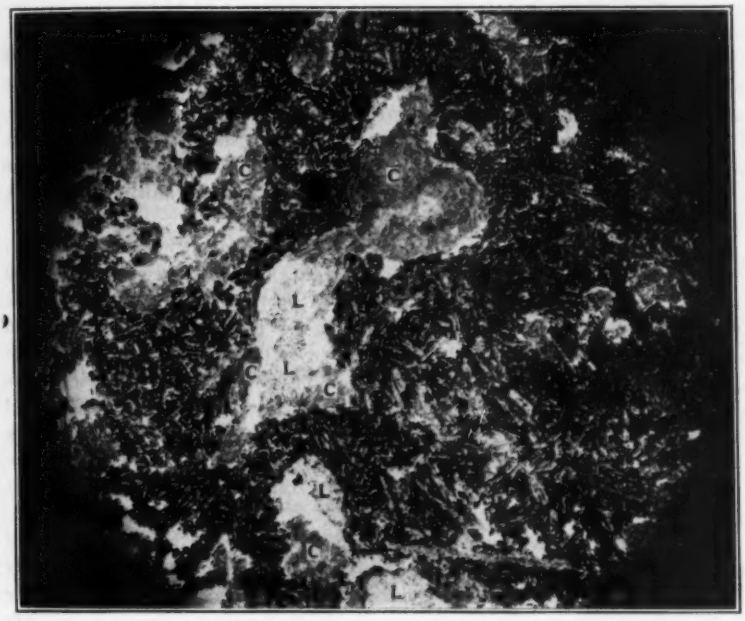

FIG. 6.—PHOTOMICROGRAPH OF ALTERED AMYDALOIDAL BASALT.

White lath-shaped grains are somewhat altered soda lime feldspar; small irregular light gray areas are pyroxene partly altered to chlorite; black is altered glassy material containing fine specks of magnetite. The large irregular areas are amydules or vesicles that have become irregularly enlarged by alterations and filled by gray masses of chlorite (C) and white masses of the zeolite, laumonite (L). The smooth white areas are holes. There is sufficient zeolite in the amydules and in minute grains throughout the rock to render its weathering qualities poor. Magnified 40 diameters.

not adhere strongly to the cement. On the other hand, both coarse and fine fragments, if thoroughly imbedded in dense concrete, are protected from the atmosphere and need cause no concern so long as their crushing strength is equal to or greater than that of the cement. Even so unstable a mineral as laumontite, which has promoted exfoliation where exposed in the copper mines of northern Michigan, is prevented from decomposing by a coating of gunite. So long as the surface of the concrete contains

only thoroughly durable aggregate and is practically weatherproof, the interior of the concrete may safely contain an aggregate of altered rock that would be condemned for monumental or high-class building stone.

Alteration at or near the earth's surface.—Perhaps the most important influence of deep-seated alteration from an engineering standpoint is to render rocks more subject to subsequent alteration at or close to the surface; that is, to weathering. It has just been shown that most of the minerals formed by deep-seated alteration are less resistant to weathering agencies than the original rock-forming minerals. The effects of weathering shown by the oldest stone buildings, however, are extremely slight in comparison with those in ledges of the same kinds of stones where both mechanical and chemical weathering agents have been acting under most favorable conditions for thousands and, in some regions, millions of years. Even the most durable rocks and minerals yield to the attack of the mildest weathering agents if the attack is continued sufficiently long, and some judgment is therefore necessary in determining the durability of a rock from a study of its weathered outcrop.

Weathering takes place mainly above the ground water table. Its principal agents the world over are water and the oxygen and carbon dioxide that have been dissolved from the air by rain. Nitrogen compounds are also dissolved from the air but are not nearly so important as oxygen and carbon dioxide. In industrial centers important quantities of sulphuric acid are derived from coal smoke. In addition to these, more carbon dioxide is derived from organic matter in the soil, and additional sulphuric acid from any pyrite that may be present in the rocks. Where pyrite is relatively abundant, as it is in most mining districts that produce nonferrous metals, sulphuric acid is by far the most important agent of chemical weathering.

Pure water has a slight solvent action on practically all minerals if they are very finely powdered, but may be considered inactive on nearly all of the original rock-forming minerals as they occur in nature. One of its principal functions in weathering is to permeate the most minute pores, and to cause swelling of well-developed micaceous minerals. This swelling continues until the minerals of the rock become pried apart and the rock disintegrates into a coarse angular sand. Small cracks are gradually developed in and around all minerals by temperature changes, and promote disintegration, especially where frequent freezing takes place, but disintegration is most marked in those granites that contain considerable mica in comparatively large crystals or clusters of flakes.

Such occurrences are abundant in unglaciated regions. One classic example is the gneissoid granite described by Merrill,[1] in the District of Columbia, where this weakening of black mica, accompanied by a small amount of general chemical composition, extends in places to a depth of 80 feet. Rock removed from this depth during tunnelling appeared hard

[1] Merrill, G. P. Rocks, Rock Weathering, and Soils, p. 185. John Wiley & Sons, 1906.

and sound at first, although it did not ring when struck by a hammer, but after a short exposure at the surface it began to crumble. It is conceivable that such a newly excavated rock might be crushed and used in concrete before it had time to show its true character, and that such a concrete unless impermeable to air and moisture would soon disintegrate.

Disintegrated coarse grained granite in Colorado makes a good surfacing for waterbound macadam and dirt roads, as rain quickly sinks through it, and any clay that it contains helps to bond it when dry; but it would be very unwise to use this material in concrete roads, at least until samples of concrete containing it had been subjected to rigid weathering and strength tests. This statement also applies to disintegrated granites in other western States.

Another important function of water is to render its dissolved oxygen and other constituents chemically active. The same constituents in dry or ordinarily humid air have no measurable effect on rock-forming minerals, with the exception of marcasite, already noted; but oxygen when dissolved in water tends especially to attack certain minerals containing ferrous iron, and carbon dioxide and sulphuric acid attack practically all minerals to some degree. Calcite and other carbonates are entirely dissolved and carried away, although the process may take place so gradually, especially in massive dolomite, that the mineral at first breaks down into a sand. The silicate minerals gradually lose some of their constituents. Black mica and chlorite first lose their iron and become bleached to resemble white mica; later they lose their iron and become bleached to resemble white mica; later they lose their magnesia and alumina and leave flaky skeletons of silica. The finely divided white mica, sericite, loses its alkali and is ultimately reduced to one of the clay minerals. The same process takes place in the feldspars and is usually faster in plagioclase than in orthoclase; but this general difference in rate is probably due largely to the fact that plagioclase in many rocks contains sericite and other alteration products which are susceptible to weathering. Pyroxene, hornblende, and olivene lose their constituents to some degree and leave an ultimate residue of clay colored by brown or red iron oxide. Pyrite as already stated, leaves a residue of brown iron oxide and contributes sulphuric acid which hastens the attack on all the other minerals. Alkalies extracted from the feldspars and micas, have some corrosive action on quartz, but quartz is the most durable of all the constituents of igneous rocks.

For this reason, after a region has undergone thorough weathering, erosion and transportation by moving water concentrate the quartz into sand. Some sands, like the well-known Ottawa sand, consist almost entirely of quartz; others where decomposition has not been quite so extreme contain minor quantities of the magnetic iron oxide, magnetite, or the iron-titanium oxide ilmenite, and muscovite; where it has been less and less extreme undecomposed remnants of feldspar and black silicate minerals are also present in the sand.

The third column of Table 2 shows that the residue remaining after complete decomposition consists of clay and earthy iron oxides which are generally distributed through clay as red, brown, or yellow coloring matter. In dry regions carbonates and sulphates of alkalies and alkaline earth (calcium and magnesium) may be present due to evaporation of ground water.

The clay minerals are the most objectionable of all that may be found in rocks, and especially in concrete aggregate. These materials form a group that differ somewhat in chemical stability, but may be regarded as essentially durable from a strictly chemical standpoint. Their tendency to absorb moisture is their bad feature. They grow in minute platy or micaceous crystals which have both a most remarkably developed cleavage and a most remarkable ability to absorb water along the cleavage planes. Chemical and microscopic analyses have identified several varieties of these minerals, all of which are white when pure. The more common varieties may be listed in two groups, the kaolin group, which includes kaolinite anoxite, and halloysite; and the leverrierite group, which includes beidellite and montmorillonite. These minerals may be present in the same mass, and the name kaolin, which is the most familiar, is likely to be applied to clays containing minerals of either or both groups. Bentonite is the name given to certain clays that consist mainly of montmorillonite and are derived directly or indirectly by the decomposition of volcanic ash.

Very little experimenting with clays is required to demonstrate that their ability to absorb water varies greatly. Some, notably kaolinite, may remain in water for days with little or no visible change and little or no tendency to become plastic; others absorb water so rapidly that they begin to slake or disintegrate immediately, and still others that contain large percentages of montmorillonite retain their absorbed water to such a degree that they swell to several times their original size. This variation is evidently due to fineness of the cleavage plates, the ability of water to reach them, and the ability of the water to adhere to the plates. The clay mineral may be so intimately mixed with amorphous silica or may be so firmly compacted that the water can not penetrate it readily; if so, fine grinding or exposure to weather for several months will develop the absorptive quality and the resulting stickiness or plasticity. Plasticity has been attributed to colloidal matter, but recent microscopic study has shown that it is due mainly if not entirely to absorption of water by the minute platy crystals. The amorphous clay minerals contain considerable water which may be partly lost by continued drying, causing them to shrink and crack. Repeated loss and reabsorption finally results in complete disintegration.

The influence of these clay minerals in promoting the disintegration of rocks is readily appreciated, but their mere presence in small quantity is not a sure sign that rapid disintegration will take place. If these minerals are thinly and uniformly distributed throughout an impervious

rock, air and water may not reach them to an appreciable degree, but if they are concentrated in bunches or layers, rapid disintegration is to be expected. Thus granites in which certain feldspar grains have been largely changed to a mass of microscopic sericite and kaolin soon become pitted owing to the weathering out of these minerals. This same granite, however, would be fairly satisfactory as a concrete aggregate, as most of it would be well inclosed in the cement and the amount of clay material reached by air and rain is very little or negligible. It should be avoided, however, if the concrete is to be subjected to much seepage. In some volcanic rocks with well developed flow structure or parallel partings, and in other kinds of rocks that happen to contain closely parallel compression or shearing cracks, these clay minerals may form along the cracks and give rise to splitting or scaling on exposure to weathering.

In general, however, rocks that contain too high a percentage of clay, and perhaps of other minerals of inferior weathering quality to be accepted for concrete aggregate give ample evidence of rapid weathering in the outcrop; but the use of weathered anorthosite, already referred to, is striking testimony that ample evidence may be ignored. In general material of doubtful quality should be exposed for some months to test its natural weathering properties, and test pieces of concrete containing it should be similarly exposed. Circumstances now and then may call for a rapid supply of aggregate where the weathering qualities of local materials are not known or are of doubtful quality; if so the mineral composition and texture of the materials considered should be accurately determined and selections made accordingly.

Products of weathering even more than those of deep-seated alteration tend to increase the amount of dust in crushed stone, and any films of the dust that persistently adhere to coarse aggregate not only weaken the bond between aggregate and cement but promote weathering where the bond is weakest.

SEDIMENTARY ROCKS.

The earliest sedimentary rocks were derived by the erosion, transfer, and redeposition of material from igneous rocks, and later sedimentary rocks have been derived from both igneous and earlier sedimentary rocks, which in part had been converted into crystalline, that is metamorphosed rocks, under intense heat and pressure. As these metamorphic rocks, with the exception of marbles, consist mainly of the same minerals as the igneous rocks and are subject to the same kinds of deep-seated and shallow alteration, it is obvious that sedimentary rocks consist of different mixtures of these minerals, their deep-seated alteration products, their final residual decomposition products, and the material that has been dissolved from them and redeposited. As there is a practically endless variation in the possible proportions of these constituents there is an equally endless variation in the weathering qualities of the rocks containing them.

Sedimentary rocks are conveniently grouped as follows:

TABLE 3—CLASSIFICATION OF SEDIMENTARY ROCKS[1]

Origin, Texture, and Structure	Name	Essential Mineral Composition
Deposited from solution by evaporation, with or without the aid of micro-organisms (crystalline to dense; massive, bedded).	Gypsum and anhydrite.	Gypsum and anhydrite alone or together; pure or accompanied by shaly or clayey material.
	(Rock salt).	Pure, or with gypsum or clayey material.
	Siliceous sinter.	Silica (opal or chalcedony).
	Travertine (limestone and dolomite in part).	Calcite (or aragonite), pure or colored by iron oxide, and divided by clayey films along the bedding.
	Iron ores (in part).	Hematite and limonite.
Deposited mainly by organic agencies, with or without modification by mechanical processes (crystalline or amorphous; fragmental, dense, massive, bedded, shaly).	Limestone. Dolomite. Flint. Chert.	Calcite or dolomite; pure or with varying amounts of sandy or shaly material, flint or chert, pyrite or marcasite. Silica, amorphous or cryptocrystalline; pure or with finely disseminated calcite.
	Diatomaceous earth and shale.	Silica, amorphous; pure or with varying amounts of shaly material.
	Coal: Anthracite. Bituminous. Lignite. Peat.	Pure or with varying amounts of shaly material and pyrite or marcasite.
Deposited mainly by mechanical processes (fragmental; dense; massive, bedded, shaly).	Gravel, conglomerate and breccia; also glacial boulder clay or till (tillite).	Rounded or angular fragments of rocks and minerals; loose or cemented by silica, iron oxide, calcite, or clayey material.
	Sand and sandstone.	Grains of quartz with or without other minerals; loose or cemented by silica, iron oxide, calcite, or clayey material.
	Clay and shale or argillite.	Microscopic clay minerals and quartz, with or without mica and other minerals; soft or consolidated

[1] Adapted from "Rocks and Rock Minerals" (by L. V. Pirsson; revised by A. Knopf).

Gypsum, though generally used to retard the setting of portland cement, is rarely if ever considered for use as aggregate because of its extreme softness and ready solubility in water. Anhydrite, though harder and only slightly soluble, is likely to be accompanied by considerable gypsum close to the surface and is therefore not to be recommended for use as aggregate. The ready solubility of gypsum and its derivative, plaster of paris, calls for some consideration of its use in concrete and cement. Although the very small quantities added to cement by manufacturers does not appreciably decrease the strength and weathering quality of the cement, excessive quantities do. The writer has heard of contractors adding considerably more calcined gypsum than permitted by specifications, to hasten the setting of cement. Such quantities are sufficient not only to weaken the cement greatly, but by dissolving to render the cement

unusually porous and to account for failure of concrete in which both cement, sand, and coarse aggregate are ideal.

Siliceous sinster is found in too few available places to be of much economic interest, but is worth mention because of its chemical durability and its porosity and low specific gravity which adapt it especially for concrete of light weight.

Travertine may be regarded as a special variety of limestone and need not be considered separately.

Limestone including dolomite is used for aggregate in greater quantity than any other kind of stone, and if it were not generally satisfactory poor concrete would be a commonplace. In other words, although calcite, the principal constituent of high calcium limestone, is slowly soluble even in pure water and rather rapidly so in rain and other waters that contain small quantities of acids, the attack on fragments that constitute coarse and even rather fine concrete aggregate is too slow to cause concern. Limestone and marble (crystalline limestone) in the most exposed parts of buildings show a slight roughening of the surface after 10 or more years, the rate depending on texture of the stone. After 25 years or more corrosion along boundaries of grains in distinctly crystalline marbles loosens them enough to cause slow crumbling, but even these corroded surfaces may be no rougher than the surface of newly exposed concrete. The appearance of weathered limestone aggregate on the surface of concrete is therefor of no consequence and it will be several centuries before any impairment of strength due to weathering of coarse limestone aggregate in ordinary buildings becomes serious, unless the concrete becomes repeatedly permeated by water from leaky roofs or other defects in the building. Where concrete with limestone aggregate is used to line aqueduct tunnels or for any structure that is continually in contact with water, seepage through the concrete is likely to corrode the limestone and thereby increase seepage or leakage to a serious degree in a short time, as in the Thirlmere Aqueduct in England.[1]

The foregoing paragraph applies to relatively pure and hard limestone or dolomite represented by Fig. 7. Porous and soft limestone varies in durability according to details of texture and composition; for example, the well-known Indiana oolitic limestone, sometimes referred to as "Bedford stone," has proved itself an excellent building stone, though rather soft for crushed stone. This stone, in spite of a considerable porosity is so well consolidated by recrystallization of both its component shell grains and matrix that it resists frost action well and is only slowly and uniformly corroded, as all limestones are, when subjected to repeated attacks of rain water. Owing to its porosity, however, it is conspicuously affected, especially if not seasoned, by seepage from the backing of a building wall, as the seepage water leaches alkali salts from cement mortar and brick, as these dissolved salts in turn leach some of the brown organic coloring

[1] Berkey, C. P., Geology of the New York City (Catskill) Aqueduct; N. Y. State Museum, Bull. 146, pp. 138-140.

matter from the stone and carry it to the surface, where it forms a brown stain which disappears after about a year's exposure to sun and air. The alkali salts themselves as well as any free acid radicles in the seepage water may be sufficiently abundant to cause a white efflorescence. Seepage due to rains while a building is being constructed may disfigure the stone temporarily without appreciably weakening it, but frequent or continuous seepage will slowly produce more serious results.

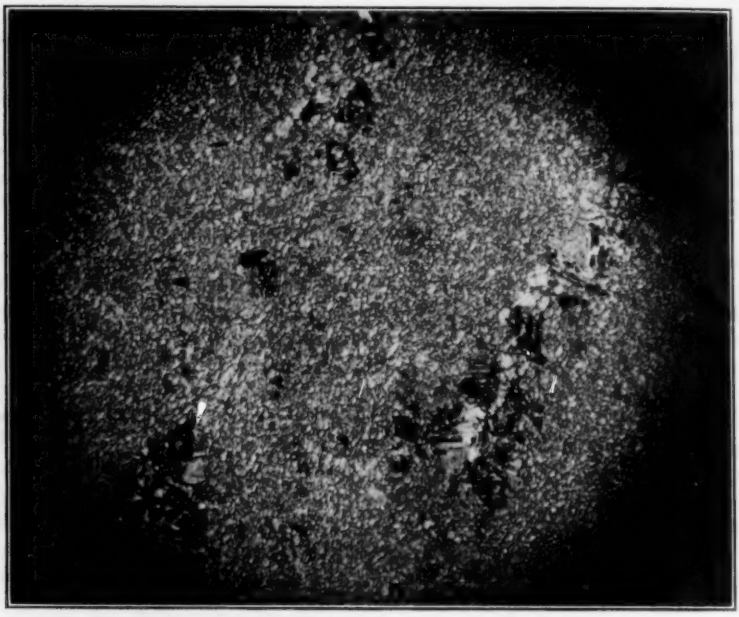

FIG. 7.—PHOTOMICROGRAPH OF FINE GRAINED DOLOMITIC LIMESTONE WITH VEINLETS OF A COARSER GRAINED CARBONATE (ANKERITE) WHICH CONTAINS CONSIDERABLE IRON.

This carbonate has been somewhat affected by weathering and its iron has partly separated as brown oxide (black in photograph). The amount of weathering in the stone as a whole is insignificant. The grains of both the stone and veinlets are firmly interlocking crystals and represent the most desirable texture of limestone or dolomite. Magnified 40 diameters.

In contrast to the good quality of this stone inferior stone may be seen at a few abandoned quarries. One sample of inferior stone from a frost-spalled wall is represented in Fig. 8. Part of this specimen appears of satisfactory quality but part of it has the original chalky matrix which for some reason has not recrystallized into strong calcite. The chalky material is so finely porous that it retains moisture and promotes frost action, and forms a relatively weak bond with the fossil shell frag-

ments. It is also a more rapid prey to chemical weathering. Were such a stone to be crushed there is further danger of minute shattering of the weakly cemented places which would decrease the strength of concrete containing the stone, and would still further promote weathering.

This inferior limestone is free from appreciable shaly or clayey materials, but the presence of these materials in limestone or sandstone is likely to be a greater cause of inferiority. The exact composition and

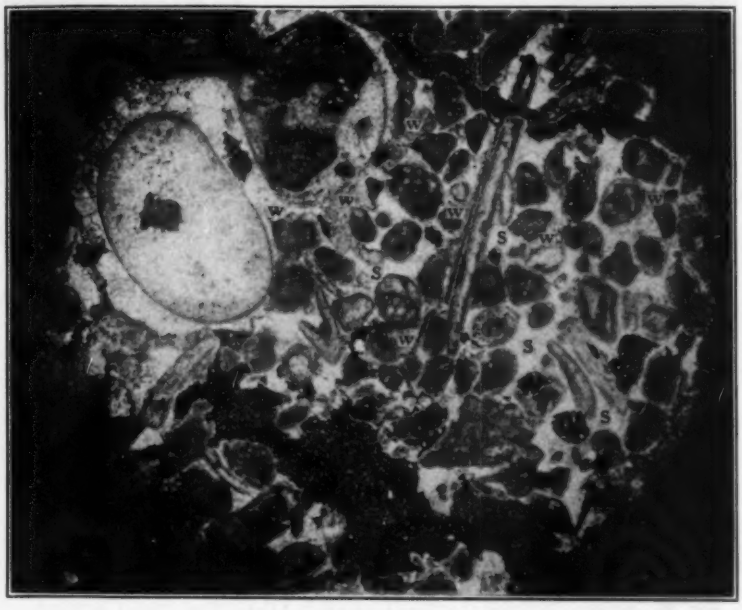

FIG. 8.—LIMESTONE COMPOSED OF SMALL FOSSIL SHELLS CEMENTED BY CALCITE.

From a condemned and abandoned quarry near Bedford, Ind. Both good and bad features are shown. Where the originally chalky cement has recrystallized into distinct grains of strong calcite (S) which is continuous with the calcite of the fossil the stone is strong and durable; but where the original weak chalky calcite (W) remains its bond with the surface of the shell is weak and it is subject to more rapid corrosion and to disruption by frost action. Magnified 40 diameters.

distribution of these materials in a stone has much to do with the rate of weathering. For example three samples of limestone from the Con-Oil Tank quarry in western Pennsylvania, submitted to the writer by Mr. P. J. Freeman were labeled "sound," "unsound," and "bad," according to the sodium sulphate test. The three were alike in mineral composition, and contained a considerable quantity of clay material of the leverrierite group

and also organic matter. These impurities were evenly distributed in the "sound" sample, which was so impervious that the sodium sulphate solution did not penetrate it appreciably. The amount of clay and organic matter present, however, are sufficient to cause suspicion that this stone if subjected to repeated wetting and drying would weather much more rapidly than the stone represented by Fig. 7.

FIG. 9.—DENSE (MICROGRANULAR) CLAYEY LIMESTONE, FROM CON-OIL TANK QUARRY IN WESTERN PENNSYLVANIA.

(Specimen furnished by P. J. Freeman.) Consists of a few small fossil shell fragments (F) imbedded in a mass of minute calcite grains which was evidently deposited as a "limestone mud" along with considerable ordinary clay mud and organic matter. The clay material has segregated in small areas which appear dark in the photograph and are indicated by straight arrows. These dark areas are cut by cracks caused by shrinkage due to loss of ground water or "quarry sap." The clay mineral is one of the leverrierite group (p. 14a) which retained a large amount of water until the rock was exposed and dried, and tended to swell or shrink as the rock became wet or dry during exposure. The organic constituent is a complex mixture of compounds a small part of which are unstable and aid in the distintegration of the rock. Magnified 20 diameters.

The "unsound" sample was intermediate between the "sound" and "bad" samples, and only the "bad" need be considered here. This sample is represented by Fig. 9, which shows clay and organic matter segregated in irregular spots or patches, which contracted so much on drying that short shrinkage cracks formed in them, and gave ready access to sodium sulphate solution as they would to frost. Such irregular spots as well

as more pronounced shaly films and streaks tend to become opened during crushing and thus to hasten disintegration.

Fig. 10 represents an impure crystalline limestone with many laminae of fine grained mica schist or phyllite which represent recrystallized layers or films of shaly matter. Recrystallization has improved the weathering quality as compared with the original shaly limestone, but the micaceous layers are so fine grained, and close together that they render the out-

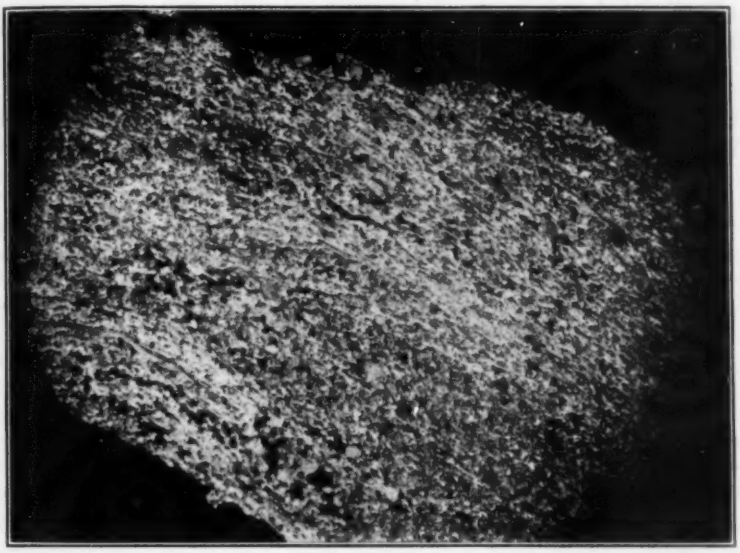

FIG. 10.—IMPURE CRYSTALLINE LIMESTONE.

The small gray grains are calcite, the small white grains quartz, and the long white streaks are linear aggregates of mica (muscovite) which envelop minute quartz grains. As these micaceous streaks become more numerous the rock grades into a phyllite (micaceous slate) or mica schist. Weathering tends to progress rapidly along these streaks and the rock close to the weathered surface is not to be recommended for use as aggregate. The streaks also cause the rock to crush into flat scales which are undesirable. (Magnified 40 diameters.) Limestones with shaly or clayey streaks and partings have similar structure but do not appear so clearly in a photograph. Rapid weathering along such shaly seams would obviously cause complete disintegration. Magnified 20 diameters.

crop more subject to weathering than is safe from the builder's viewpoint, and furthermore cause the stone to crush into thin scales.

As fine grinding increases the rate of corrosion, the use of limestone screenings deserves special consideration. The value of pulverized limestone added to soil is due to its ready reaction with acids in the soil, but for this reaction to take place completed within a season grains must be as small as 200 mesh, and within 4 or 5 years at least 50 mesh. It may be inferred therefore that limestone screenings in concrete that are

out of contact with the soil and not subjected to seepage due to faulty construction are not objectionable from the standpoint of weathering except on exposed surfaces, where the smallest particles will weather out rapidly, and larger particles may become loosened after one or more years by corrosion along their contacts with the cement matrix. Even grains or chips a quarter of an inch in diameter, such as may be used as "pebble dash" or as facing on artificial stone are likely to become loosened by corrosion after a few to several years, the rate depending upon the degree of permeability along their boundaries and the amount and composition of rain water that comes in contact with the concrete.

Dolomite, or high magnesium limestone and marble, is much more slowly corroded than high calcium limestone, though the effects of corrosion are similar. Pulverized dolomite is also used successfully to correct the acidity of soils, and is therefore readily soluble in extremely fine grains; but grains as coarse as ordinary sand are very resistant to the natural weathering conditions that effect ordinary buildings.

Flint and chert consist entirely of silica and are immune to ordinary weathering if pure. Some flints and cherts, however, contain considerable quantities of finely divided calcite, which is leached out of weathered outcrops and renders the silica porous, or honeycombed. If calcite is very abundant the weathered flint or chert may fall to a sand or powder. This kind of weathering however takes place so slowly as to be no cause for worry to users of concrete. The worst that can be expected is for pieces of aggregate exposed at the surface to become finely pitted after considerable exposure, and for some internal corrosion to result from seepage, though to a less degree than with limestone.

Cases have been cited where "chats" or finely crushed chert tailings produced in the milling of zinc ores in the Missouri-Oklahoma-Kansas mining region have given poor results when used as concrete aggregate. If the chert were pure, the poor results could not be attributed to its poor weathering qualities; but if it were accompanied by finely divided pyrite, or zinc sulphide, the oxidation of these minerals would cause corrosion, and by attacking any calcite in the chert or the lime in the cement would form gypsum, which by crystallizing would tend to expand and disrupt the cement, and later by weathering out, would leave the cement to crumble. As no samples of inferior chert have been seen by the writer a more specific statement can not be made. The material used in any structure of poor concrete as well as its source of supply should be examined before a more definite explanation of the failure can be given.

Diatomaceous earth, if pure, is composed of porous silica and is both durable and useful in lightening the weight of cement mortar and concrete. In California there is an extensive formation of consolidated diatomaceous earth called diatomaceous shale. It is in part practically pure silica and would serve as a light weight aggregate, but in part it contains considerable true shale and is subject to the same criticism as shaly rocks in general.

Coal would not be mentioned except for the presence of fragments of soft coal and lignite in certain sands and gravels that have given poor results in concrete. Anthracite and low volatile bituminous coals are not appreciably affected after considerable exposure. High volatile coals deteriorate more rapidly so far as fuel efficiency is concerned owing to loss of gaseous content, but disintegration is not very marked. Lignites, however, contain as much as 33 per cent of water which is rapidly lost by evaporation, and causes the original solid material to fall to powder. If lignite could find its way into concrete before loss of water were possible, and the concrete were to remain below water, no trouble would be expected unless the lignite were so abundant that its low crushing strength became a serious matter; but in concrete exposed to the air lignite would be subject to slow drying and crumbling. Small to considerable quantities of marcasite may be present in soft coal and lignite, and as already shown is likely to decompose rapidly and attack the cement, but it has probably been thoroughly oxidized in the small fragments that are found in gravels.

Gravel or unconsolidated conglomerate, may consist of pebbles of any kind or kinds of rock and may be accompanied by very little or much sandy and clayey material. Some well sorted gravels consist entirely of quartz pebbles with intersticial filling of clean quartz—constituents that are immune to ordinary weathering. Others are similar but contain sufficient clay to require careful washing. Still others contain a variety of pebbles including shale and less common rocks of inferior weathering quality. In general, gravels that have been derived by long continued erosion and by continued reworking of sediments or from regions containing extensive areas of durable rocks show the least variation in composition and consist of the most durable rocks and minerals. Other gravels, formed in regions of diversified rocks and not subjected to reworking contain considerable quantities of even the least durable rocks. Not only pebbles of shale, but even of clay (mud balls) may be present. This statement applies whether gravels have been derived by stream action or by glacial action modified by stream action. It applies less to marine gravels. With the origin of gravels in mind, it is obvious that one region may be characterized by gravels that are entirely excellent, another by gravels that are poor, and still another by both good and poor deposits of gravel.

In the North Central States gravels were deposited during both early and late stages of the glacial period. Those formed during early stages were subjected to prolonged weathering during interglacial stages and pebbles of hard granitic rocks became disintegrated, whereas those formed during the last glacial stage are too recent to be appreciably weathered. In the southern Coastal Plain region, beyond the limits of glaciation the older gravels of Cretaceous and Tertiary age, are considerably disintegrated. Even pebbles of so durable a rock as quartzite have been appreciably affected, and only pebbles of vein quartz remain practically intact. Some of the recent gravels in the South and elsewhere may contain so much

clay that excessive washing is necessary to render it fit for use. Determination of the mineral contents of any newly opened gravel deposit should be made before it is used, and if any doubt exists regarding its durability, the gravel should be given a weathering test before risking it in exposed parts of concrete.

Conglomerates, as the name is usually understood, range all the way from weakly cemented gravel to some of the very hardest and most impervious rocks. They obviously range as widely in composition as gravels.

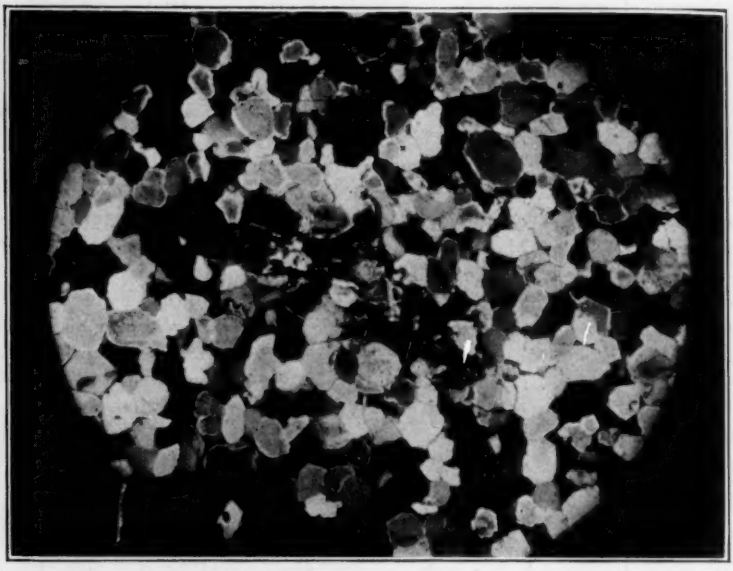

FIG. 11.—PHOTOMICROGRAPH OF SILICEOUS SANDSTONE.

Composed almost entirely of quartz grains some of which appear dark because of the position of their crystal axes with respect to the crossed nicol prisms of the microscope. They are cemented by additional quartz which has enlarged the grains and caused them to interlock as in a crystalline rock. Magnified 40 diameters. This sandstone should crush without yielding an excessive amount of screenings, and the crushed fragments should bond well with cement.

Those with clayey or shaly matrix rapidly disintegrate into loose gravel, and only their thoroughly disintegrated parts, after thorough washing, are worthy of consideration for concrete aggregate. Those with calcite as the principal cementing material, are also subject to rapid weathering in the outcrop, but will serve well as aggregate with the same limiting qualifications already made for other rocks containing calcite. Conglomerates with iron oxide or silica as the principal matrix and without a noteworthy per cent of shale pebbles are durable. In short the percentage and composi-

tion of the matrix are the most critical factors in determining the value of a conglomerate for concrete aggregate.

Some conglomerates originally with poor matrices have been rendered durable by deep-seated alteration; for example, the well-known Roxbury puddingstone in and near Boston originally had a matrix of sandy clay, which by alteration was recrystallized into interlocking grains of quartz and fine mica, the latter well protected by the quartz. The recrystallized matrix is welded to the pebbles and is nearly equal to them in crushing strength.

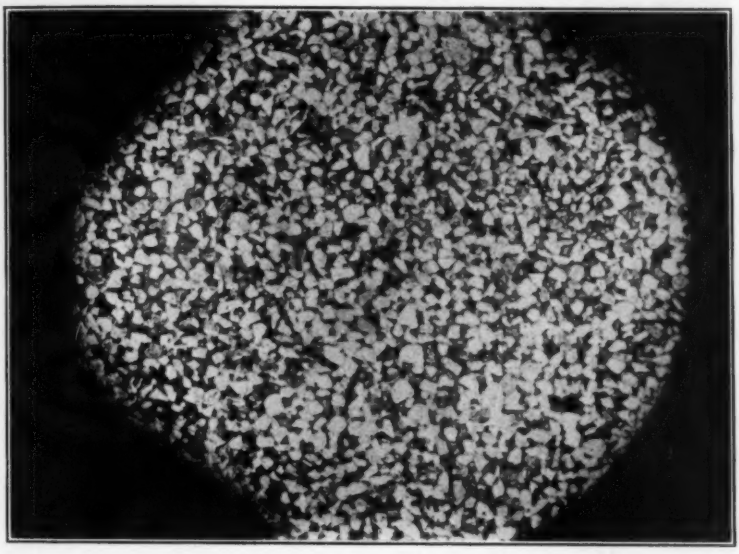

FIG. 12.—PHOTOMICROGRAPH OF SANDSTONE COMPOSED OF WHITE GRAINS OF QUARTZ AND FELDSPAR CEMENTED BY (GRAY) CALCITE.

This stone is much softer than that shown in Fig. 11, but if crushed and well screened should be satisfactory in concrete that is not subjected to seepage. Like all stones containing much calcite it becomes noticeably corroded after many years of exposure to rains.

Sandstones differ in general from conglomerates only in the size of constituent fragments, and are identical in range of composition, degree of consolidation, and variation in weathering qualities. They are illustrated by Figs. 11-15, which are adequately explained beneath their titles.

Clay and its consolidated equivalents, shale or argillite, are the most harmful and most widespread of the natural minerals that may be found in concrete aggregates. Their bad behavior is so familiar that deposits containing conspicuous quantities of them are promptly condemned; but they may also occur as inconspicuous, thin beds, streaks, or films, in all

other kinds of sedimentary rocks, and these streaks or films may be sufficiently abundant to give to these rocks weathering properties like those of shale. The weathering properties of clay minerals have already been considered. Those of shale are similar but not so quickly developed. The consolidation of clay into shale, due to the weight of overlying strata is accompanied by a squeezing out of much of the water originally present; but if the minute platy structure of the clay minerals is preserved, considerable moisture will be retained until the shale is exposed, when its

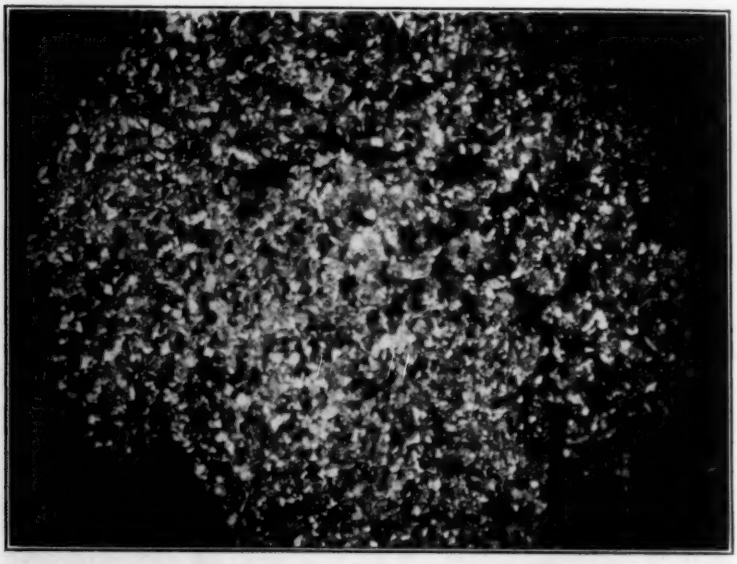

FIG. 13.—PHOTOMICROGRAPH OF A FINE GRAINED SANDSTONE.

Composed of angular white grains of quartz and fresh feldspar (microcline) and gray grains of weathered feldspar (plagioclase). The weathered feldspar is not readily distinguished from the gray matrix which consists largely of clay minerals of the leverrierite group accompanied by some halloysite of the kaolin group. This sandstone weathers rapidly and yields so much clayey dust on crushing that it should not be used in concrete unless, after thorough drying or seasoning it is used in concrete for interior walls where superior strength is not required and where there is no danger of seepage. Magnified 20 diameters.

evaporation will favor disintegration. Where such prompt disintegration does not take place, alternate wetting and drying after exposure will soon bring it about, and frost action will accelerate it. Some layers of the shale may contain more of the platy clay minerals than others, and tend to form partings along which splitting takes place after brief to moderate exposure. Other layers consist mainly of minute grains of quartz, which, if cemented mainly by silica, makes a very durable rock. If the shaly or finely layered structure is not conspicuous, these clay or mud stones are

more aptly termed argillites, some of which are comparatively durable and others not.

Deep-seated alteration may improve shale and argillite just as it may the clayey or argillaceous conglomerates and sandstones by converting the clay minerals into interlocking crystals of quartz and mica or the

FIG. 14.—COARSE UNEVEN GRAINED SANDSTONE IN WHICH THE MATRIX, ORIGINALLY OF CLAYEY MATERIAL, HAS BEEN RECRYSTALLIZED DURING DEEP-SEATED ALTERATION INTO AN INTERLOCKING MIXTURE OF QUARTZ AND SERICITE.

This quartz has grown on to the original quartz grains, enlarging them and causing them to interlock to some degree. The sericite is so enveloped in quartz that it does not detract appreciably from the stone's quality as concrete aggregate. In weathered outcrops, however, the sericite may have become swelled to some extent, or may have changed back again into clay minerals. Only the unweathered stone should be used. Magnified 40 diameters. The dark grains are of quartz so situated that they appear dark between crossed nicol prisms.

aluminum silicates that characterize certain metamorphic rocks. Thus the argillite or slate formerly quarried extensively around Boston has resisted the weather excellently in old buildings for 75 years, and has been used as crushed stone with good results. Although somewhat softer than crushed igneous rocks in the vicinity, it is quite satisfactory as con-

crete aggregate. Argillite or hornfels in eastern Pennsylvania and that at Little Rock, Ark., have been so thoroughly baked by igneous intrusions that they also have completely lost their original argillaceous character and are nearly as hard and tough as diabase, which they resemble at first glance, and are even more resistant to prolonged weathering. They are used extensively as crushed stone, and have been commonly referred to as "trap rock." These examples show that rocks of shaly origin are not everywhere of inferior weathering quality, and it may be repeated

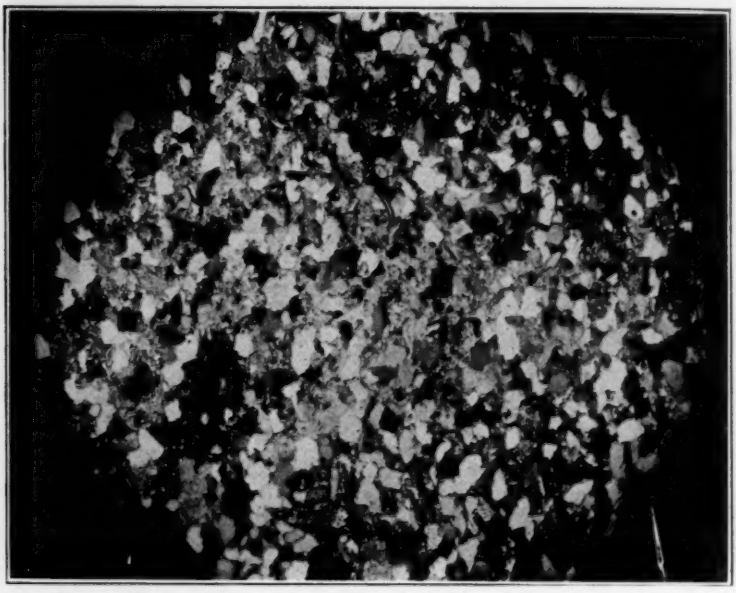

FIG. 15.—PHOTOMICROGRAPH OF SANDSTONE OR "GRAYWACKE."

Composed of interlocking grains of quartz and feldspar and a few small jet black grains of magnetite in a matrix that has recrystallized into light gray calcite accompanied by considerable fibrous hornblende. The hornblende tends to border the quartz grains but does not show clearly in the photograph. This stone, like the altered diabase (Fig. 4) and other rocks containing much calcite is suitable for aggregate in concrete that is not subjected to seepage.

that rocks for concrete aggregate must be judged not by their commercial names but by their mineral compositions and textures, and the results of physical tests. The stone quarried at one of the quarries at Little Rock is illustrated in Fig. 16. It should be borne in mind, however, that the good quality of rock produced from originally poor rock by this process is of limited distribution and is likely to grade into rock of rather similar appearance that has been only partly changed and still possesses enough of its original poor quality to be condemned.

METAMORPHIC ROCKS.

The processes of alteration that converted the clayey rocks just considered into durable rocks may be called mild metamorphism. More intense metamorphism would have developed visibly crystalline textures and perhaps minerals characteristic of extremely high temperatures such as garnet and cyanite. Local metamorphism along the contacts of igneous

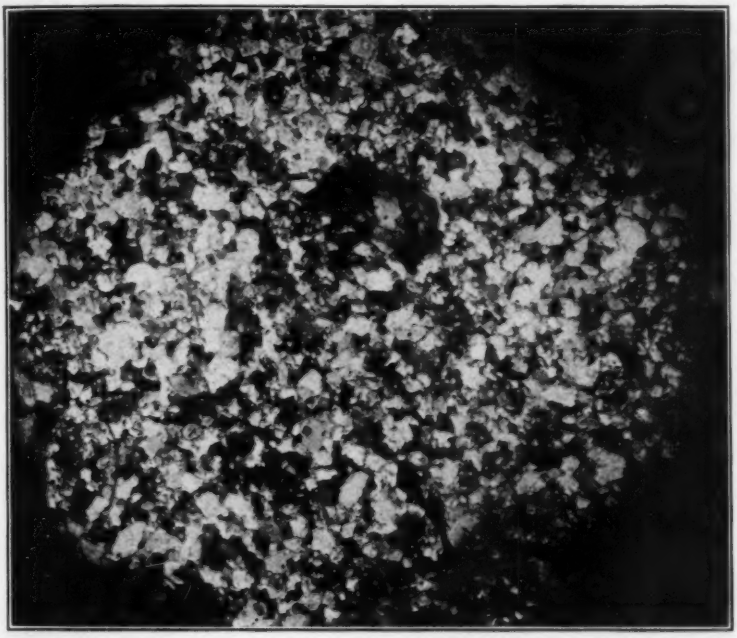

FIG. 16.—HORNFELS OR "BAKED SLATE," LITTLE ROCK, ARKANSAS.

It consists of irregular interlocking quartz grains. Pressure and heat from the adjacent igneous intrusion permitted the siliceous material of the original matrix to combine with original quartz grains and develop this interlocking crystalline texture, and at the same time to eliminate pore space. The only other mineral present in noteworthy amount is white mica (sericite) in isolated fine grained aggregates, none of which is more than one-tenth of a millimeter in diameter, and all of which are so well inclosed by the interlocking quartz grains that they are protected from weathering agencies. Magnified 40 diameters.

rocks produces massive rocks like the baked argillites or hornstones of eastern Pennsylvania and Arkansas, but regional metamorphism resulting from the combined effects of extensive igneous intrusion and simultaneous folding of the earth's crust, produces rocks whose minerals have a parallel foliated or schistose arrangement. Most of the foliated or schistose rocks are similar in mineral composition to the more common igneous

rocks, although the proportions of minerals may differ. These minerals may merely have undergone rearrangement during metamorphism or they may be the products of reaction between two or more minerals that became unstable under the conditions of metamorphism.

The most common metamorphic rocks belong to two groups, the gneisses and schists, according to the character of their foliation. Gneisses are coarsely foliated crystalline rocks and are usually similar to granite in composition. They are commonly derived directly from granite or from

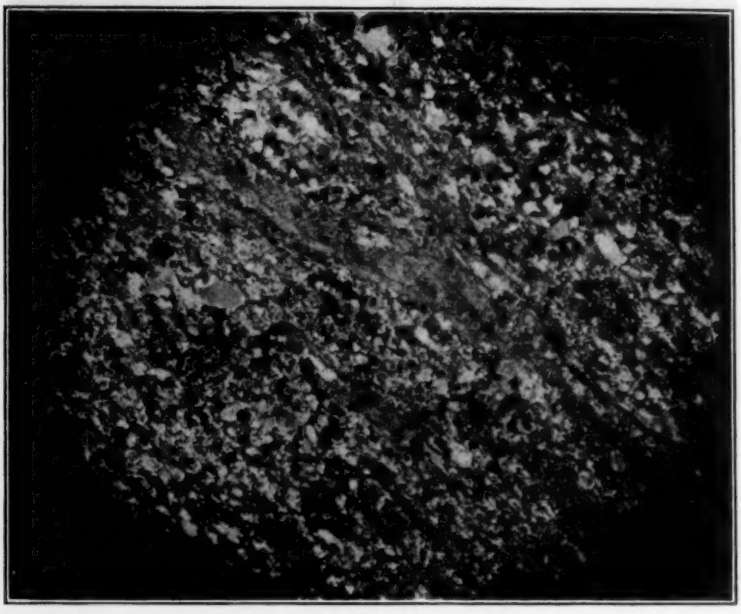

FIG. 17.—MICA SCHIST, FINELY FOLIATED.

Composed of white grains of quartz and gray streaks of mica shreds. It will crush into flat scaly fragments. The highly micaceous layers especially after crushing are likely, if exposed, to absorb water and cause disintegration, especially if they are even slightly weathered. Magnified 40 diameters.

feldspathic sandstone or by the injection of stringers of granite along the foliated planes of any crystalline rock. Less common varieties are derived from igneous rocks other than granite.

The schists are finely foliated crystalline rocks and contain large amounts of platy or foliated minerals such as mica, chlorite, and talc, and different varieties of hornblende which are foliated to a less degree. Mica schists are commonly derived from shales and shaly sandstones and less commonly from volcanic rocks. Besides their principal minerals, mica and quartz with or without feldspars, they may contain one or more of

the following minerals in considerable quantity; garnet, staurolite, andalusite, cyanite, sillimanite, and tourmaline, all of which are very resistant to weathering. Hornblende schists are derived from igneous rocks, notably gabbro and diabase, that contain large quantities of pyroxene or hornblende and also from impure magnesian limestone. Chlorite schists are mostly derived from the same rocks under less extreme conditions. Talc schists or soapstones are derived from both igneous rocks and limestones that contain large quantities of magnesia. Talc is very durable chemically, but is extremely soft and slippery and therefore not desirable in concrete aggregate.

FIG. 18.—COARSELY FOLIATED MICA SCHIST.

Round single grains or aggregates of quartz are enveloped by wavy streaks of mica. The coarse and irregular grain and wavy structure reduces the tendency to crush into thin scales, and the rock if free from weathering will be satisfactory in concrete, though less desirable than massive igneous rocks and limestones.

Further discussion of the minerals of the gneisses and schists would be merely repetition of pages dealing with the minerals of igneous rocks. The outstanding feature from the standpoint of weathering is the foliated structure. The more finely foliated a rock is, the greater is its tendency to crush into small scaly fragments undesirable for concrete aggregate. Concentration of the micaceous minerals along foliation planes, furthermore, tends to promote disintegration. (See Fig. 17.) The comparatively small amount of mica in gneisses and some schists renders them less subject to disintegration than the highly micaceous schists. Unusually fine-

grained schists that appear to consist entirely of mica may exfoliate appreciably after a few months of exposure in humid regions. The beginnings of weathering may extend a considerable distance below the surface of outcrops, and rock that looks satisfactory when newly quarried may soon begin to disintegrate. This condition, already referred to, is more likely to occur in foliated than in unfoliated rocks, and should be borne in mind, especially in non-glaciated regions, where weathering agents have had the greatest opportunity to become effective. Coarsely foliated rocks

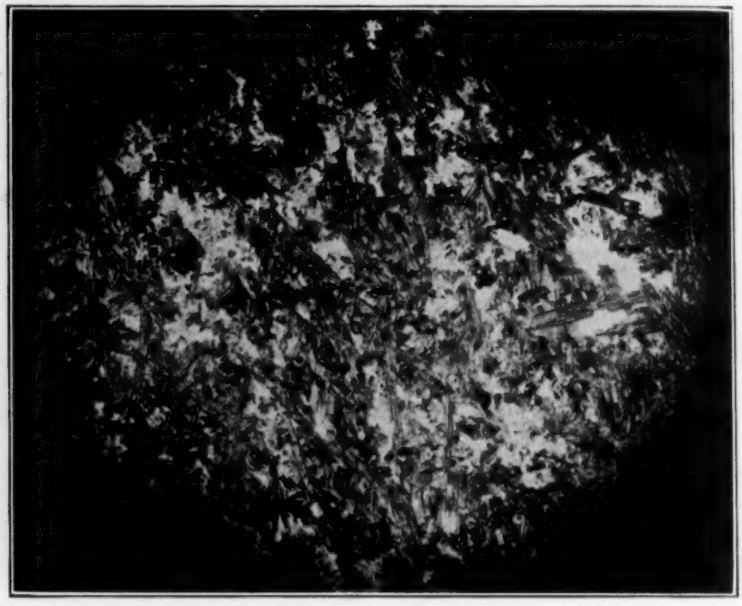

FIG. 19.—HORNBLENDE SCHIST.

Composed of irregular white grains of soda-lime feldspar in a gray network of hornblende crystals. It is almost free from foliation and practically free from weathering effects, and should make a satisfactory aggregate.

that crush into clean, angular fragments and are free from appreciable weathering effects when quarried (Figs. 18 and 19) need cause no worry over the weathering qualities of concrete containing them, and even the somewhat weathered stone should endure within concrete that is protected by an adequate mortar coating of cement and quartz sand. Screenings from even the slightly weathered gneisses and schists, however, should be rejected.

Unfoliated metamorphic rocks include marble, quartzite, and several unusual kinds, any of which may grade into foliated rocks. Pure marbles

have been considered with limestone and dolomite. Impure marbles grade into massive or schistose rocks composed largely of certain pyroxenes, hornblendes, and garnets, minerals which are chemically durable themselves, but may be accompanied by pyrite and other sulphides, and should therefore be carefully examined before being used. Quartzites are impervious rocks consisting essentially of quartz and derived from quartzose sandstones. It is one of the hardest and most durable of all rocks. A little mica is commonly present, and increasing quantities of it mark a gradation into mica schist. These less pure quartzites disintegrate after prolonged exposure either in ledges or in gravels.

CONCLUSIONS.

This general review of the weathering qualites of natural rocks used as concrete aggregates shows that most of them, if free from weathering, are satisfactory; that certain minerals, particularly the clay group and certain zeolites are very objectionable and that others including micas concentrated in fine grained flaky masses and calcite in finely disseminated grains among other minerals may promote disintegration under certain conditions; that certain textures and structures, notably fragmental texture and shaly structure in sedimentary rocks, flow structure in volcanic rocks, and highly schistose structure in metamorphic rocks, aid in the disintegration of rocks that are mainly composed of durable minerals. The weathering qualites of rocks as revealed by their outcrops and by exposure in old stone buildings should be interpreted with due regard to conditions existing in concrete construction. Stone that weathers slowly on exposure may be satisfactory when imbedded in cement, but fine aggregate of the same stone should not be used in coatings of cement mortar exposed to the weather; neither should rocks containing appreciable quantities of slowly soluble minerals be used in concrete that is subjected to seepage of ground water. The same slowly soluble minerals may be corroded by seepage through defective walls of buildings.

Some of these critical factors may be adequately judged by the contractor or engineer, and others may require special examination by the petrologist. Co-operation of the three will give the best results, especially where deposits of hitherto untried material are to be exploited.

Discussion—Qualifications of Stone for Aggregate.

Mr. Kriege.

H. E. Kriege (*By Letter*).—In reference to flints and cherts—I would be very happy to have an expression from Mr. Loughlin regarding any mineralogical differences which have come to his attention in the study of these materials. He is no doubt familiar with preliminary investigations which were carried on in the laboratories of the Missouri State Highway Commission on the "cherts" occurring in the Burlington formation in that state. An article was published by F. V. Reagel, in the latter part of 1924, regarding some of the observations which had been made, both in the field and in the laboratories with these cherts. I was interested, at the same time, in some physical, chemical and microscopic examinations of these cherts, to determine if possible what characters were responsible for the peculiar behavior in the disintegration of this chert material. I was unable to carry to completion the investigation at that time.

The siliceous material associated with the zinc ores of the Southwest Missouri district were found to make excellent aggregate for concrete. The sulphate content of the partially oxidized zinc sulphide ore remaining in the chats was very high and was readily soluble in water. It would seem as though the sulphate content at the surface of the chat particles would be in the most advantageous position for interference with the cementing properties of the cementing materials. However, some of the strongest concrete cylinders in the Missouri State Highway laboratories during my connection there were made using these chats. I would be interested to know if anything has come to Mr. Loughlin's attention which might throw some light on explanation of this apparent digression for our general knowledge of concrete.

In the photograph shown of a trap rock from Little Rock, Ark., it was stated that the matrix surrounding the crystals consisted largely of crystalline clay material. I would be very interested to know as to what mineral this is identified.

We are at present engaged in an investigation of the physical, chemical, and mineralogical properties of limestone in Northern Ohio, Southern Michigan and Eastern Indiana, attempting to relate these properties with their behavior in concrete, in bituminous and waterbound macadam filter beds, etc. We trust that some light will be shed on this type of material for concrete aggregate, which will be beneficial to the industry at large.

Mr. Loughlin.

Mr. Loughlin.—As I have not made a systematic study of cherts I cannot add anything more specific than the paragraphs on flint and chert in my paper. It would be most interesting to supplement Mr. Reagel's tests with microscopic examinations of the cherts used. Mr. Kriege's comment that chats containing considerable quantities of zinc sulphate made excellent concrete should be followed by quantitative chemical, and if necessary, microscopic examination to determine what becomes of the zinc and

the sulphate radical. During the oxidation of ore deposits it is common for zinc sulphate to react with limestone and to be deposited as zinc carbonate or basic carbonate while the lime is removed as sulphate and may be finally deposited as gypsum. The same reaction would be expected with any free lime or calcium hydrate present in the cement. Hydrous zinc silicate is also formed during the oxidation of ore deposits, but whether reaction between zinc sulphate and the silicate constituents of cement could take place appreciably during the setting of cement is not known. The presence of zinc carbonate and silicate in cement would presumably do no harm, but the danger of an excessive amount of gypsum needs no further comment.

Mr. Kriege's question regarding the "trap rock" from Little Rock, Ark., is answered in the explanation of Fig. 16, which because of scant time I treated too hastily when presenting my paper.

Mr. Jackson.

F. H. JACKSON.—One underlying thought seemed to run through the paper, and that was the fact that as long as the aggregates were protected from the action of moisture and air, a great many of them are satisfactory even though they might weather quickly if the concrete is exposed to the weather. It seems to me that this emphasizes the fact that we must pay the greatest attention to the details of the manufacture of our concrete. I think that we can frequently protect some aggregates which might be somewhat questionable by taking unusual precautions in the details of manufacture and thus provide not only for high strength concrete but dense and impervious concrete as well.

There is just one other point: Those of us who are building concrete pavements and bridges are naturally much interested in the durability and permanence of concrete, because highways and highway bridges are undoubtedly subjected to as severe weathering conditions as any type of concrete structure. We have endeavored to safeguard our aggregates by the use of the so-called sulphate test for soundness, which has proved of considerable value, I believe, even though it has been shown in some cases to be somewhat severe. The Bureau of Public Roads has, for the last two years, been conducting quite an extensive investigation on the resistance of concrete to repeated frost action. We have under investigation eighteen varieties of coarse aggregates, most of which fail in the sodium sulphate soundness test, and I think that within a short time we will have considerable practical information along this line.

Mr. Johnson.

N. C. JOHNSON.—I would like to call attention to another aspect of this discussion—the effect of decomposition of products in the aggregate on the ultimate value of the concrete. There are many rocks which form decomposition products in the presence of moisture, and all concrete, unless it be in a steam-heated room, is permeated with moisture and is permeable to moisture to a degree which I doubt many realize.

The making of concrete and the use of rock as aggregate is essentially a marriage between the hydration products of cement and the rock. It must be mutual. So many rocks refuse this marriage that we often have

in concrete a mere encasement of aggregate by mortar; and in such concretes durability is limited.

In commercial work, each of us is confronted with the practical impossibility of securing unquestionably superior rock. You even run against a stone wall in trying to get a definite size aggregate. You must either take what the local agent or oftentimes the local ring is willing to give you, or you can go outside and import it at a prohibitive price.

Also no quarry or gravel bank produces rock of a definite and fixed quality. It is stratified and variable in nature. I know of one quarry in New Jersey that is furnishing about one-third trap rock and two-thirds indurated black clay that lies confined between two of these volcanic trap-rock dikes. The clay goes bad very quickly, but not—and I would emphasize this point—not in 28 days, yet these dealers point to strength tests at 28 days as proving that this stuff is good, when everybody knows by later results that it is not good.

If we are going to have endurance, if we are going to live up to our advertising and have concrete for permanence, we must give thought to these things. If we continue to judge all our concrete on the basis of strength at 28 days in a laboratory test, progress is stopped. We must find a better test; we must find a better acceptance test, and the geologist can do a world of good to help us if he will tell us in any given locality, for any given surface, what the decomposition products are going to be and then we can find out what effect they will have on cement and on the concrete in which a given rock is used.

Mr. Ries. H. RIES* (*By Letter.*)—Dr. Loughlin has presented a most interesting paper on the qualifications of different kinds of stone for concrete aggregate, and emphasizes the need for giving attention to a matter which is of vital importance to the durability of concrete.

It would be of great value if someone could collect data on the different types of rock which have been used, the conditions of use, and the durability of the concrete in which they were used as aggregate.

A similar set of data might also be collected for sand, for it is a well-known fact that the mineralogical composition of sand shows considerable variation, and is not always composed of quartz grains as some people believe. It so happens that on my own property I have two concrete walks, both laid at least 20 years ago. The sand used in one was crushed granite, and the concrete is as sound today as when laid. The sand in the other is local natural sand, and contains a number of calcareous shale particles. A number of the slabs have developed cracks which are filled with veinlets of lime carbonate.

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Portland Cement in Concrete Engineering.*

By R. H. Bogue.†

To the concrete engineer, portland cement is only one of a number of ingredients which together go to make up concrete. In addition to portland cement, there are added some form of coarse aggregate such as gravel, crushed rock, slag or other material; some form of fine aggregate such as sand or pulverized rock or slag; and water. Occasionally still other materials are added with the intent or hope of imparting some special property to the product. These are sometimes useful; sometimes not. Thus lime, gypsum, cellite, trass, puzzolana, ground slag, calcium chloride and various other materials have found their way into concrete.

Many of these materials are not definitely standard. Almost every kind of igneous and sedimentary rock has been used as coarse and as fine aggregate. The quality and purity of the admixtures vary over a wide range. The water used contains whatever salts happen to be present in the local supply, and even sea water has been used. Of all of the materials that go into concrete, portland cement is probably the most constant.

Engineering Research on Materials.—It is not surprising that the engineer who is responsible for the making of concrete has concerned himself primarily with such factors as are most variable, and as he can most effectively control. It was learned a great many years ago that soft and easily friable or unsound stone, used as aggregate material, imparted an element of uncertainty and probable weakness to the concrete. It has long been recognized that the ratio of the fine to the coarse aggregate could not vary over too great a range without grave danger to the permanence or resistance of the concrete. The seriousness of too much water in mortars was known even to Vicat. In 1818 this pioneer in research on mortars wrote: [1]

> "The whole secret of good manipulation and right employ is condensed in the following precept: 'Stiff mortar and materials soaked.' Our bricklayers, on the contrary, seem to have taken for their motto: 'Dry bricks and drowned mortar.'"

Throughout a great part of the period during which portland cement has been manufactured, serious-minded investigators have recognized the problems of correct proportioning, of intimacy of mixture, of fineness of grinding and of water ratio as of prime importance. But it has been left to the progressive engineers of the present generation to discover and apply the laws which control these important factors.

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† Research director of the Portland Cement Association fellowship at the Bureau of Standards. This fellowship consists of a staff of physical chemists, physicists and petrographers conducting research on portland cement under the joint support of the Portland Cement Association and the National Bureau of Standards.

[1] "A practical scientific treatise on calcareous mortars and cements, artificial and natural," translated by J. T. Smith, 1837, p. 94.

This has been done within the last few years and as a consequence these laws are only now being tested on large scale operations and are only now being incorporated into general practice in concrete construction. The extreme importance attached to such problems has caused the engineer to devote his attention, up to the present time, almost exclusively upon the mechanical features of concrete engineering.

Reaching Towards 100 Per Cent Efficiency.—But this is an age of advancement. The cry for new knowledge is in the air. President Coolidge has emphasized repeatedly the consciousness which is growing within us that no matter how satisfactory the present may appear, we must hold our eye to an even higher level of attainment in our striving for the ultimate perfection. As he has put it,[1]

> "The hope of tomorrow lies in the development of the instruments of today. The prospects for advancement lie in maintaining those conditions which have stimulated invention and industry and commerce. The only road to a more progressive age lies in perfecting the instrumentalities of this age."

Questions are being asked now in all seriousness which a few years ago would have been regarded as absurd because unanswerable. The solution of the age-old problem of correct proportioning has led to a thirst for further progress in whatsoever manner might be necessary towards the goal of 100 per cent efficiency for the structures of tomorrow.

This does not mean that the accomplishments of today are poor or even that they are not entirely satisfactory in a great proportion of cases. On the contrary, as has been pointed out at the sessions of this Institute, the user of concrete is able today to build with greater assurance of entire satisfaction than ever before in history. This is brought about largely by an understanding of the laws of correct design.

Now this is the point I wish to emphasize. It is within our very present experience that we have learned the importance of conforming to the laws of scientific control in making concrete. We are very conscious of the reality of the benefits obtained by following those laws. All of this brings us to the belief that an unravelling of the laws governing the constitution of cement clinker and the behavior of the constituents of cement when made into clinker may hold developments which today can only be sensed but not definitely apprehended. In the history of industry there are cases too numerous to count wherein scientific knowledge has given to that industry, or to some part of it, some measure of added strength. It may have taken the form of substituting an intelligent and certain control of operation for an unintelligent or uncertain rule-of-thumb control. It may have resulted in some change in the procedure which affected the cost of operation, or the composition limits of available raw materials, or the rate of production, or the quality of the finished product, or the market, or the proper utilization of the product. Just in what department our investigations will prove to be of greatest value, we do not know, but we are convinced that scientific information on the nature of the compounds

[1] Address before Amherst College Alumni, Boston, 1916.

of clinker, and the reactions of these compounds, holds promise of application in many directions. Perhaps the eventual direct application may be in manufacture—in raw material—in control—or it may be in the utilization of cement in concrete. But a benefit in one department is reflected into all of the other departments. An improvement in any branch of manufacture means also an advantage in the final concrete.

This is one of our ambitions; one of the reasons for our existence. The other is to extend the field of usefulness of concrete by the development of a material which will possess to a higher degree the virtuous qualities for which concrete is now well known. Or even, perhaps, to go beyond these to the opening up of possibilities in construction or art or industry which are not yet conceived. In the case of many commodities, supply follows the demand, but in the case of the new discoveries of science, a market is born where none existed and the creation of the product opens to reality trails of progress, of achievement, which formerly were lighted only by the Aladdin's lamp of imagination.

The Problem of Portland Cement.—Our problem, therefore, is how most effectively, most directly, to project our experimentation to the accomplishment of these ends. Much research has been undertaken on portland cement (see Paper No. 3 below) but, with a few notable exceptions, these investigations have contributed little to the solution of the fundamental problems of constitution and behavior. The experimentation has been conscientiously undertaken but has been inadequate for important generalizations. Thus the behavior of a given cement, or a number of cements, when subjected to a given set of conditions may be of importance, but it cannot suffice for such generalizations as will affect the industry as a whole. A discovery of the laws which govern the relation between constitution and cement value can be made only with the pure materials and the employment of the most refined methods of physico-chemical measurements.

In the few exceptions referred to above, this condition has been appreciated. The contributions of Rankin and his associates at the Geophysical Laboratory, and of Bates and his assistants at the Bureau of Standards have opened the way to the comprehensive studies which we are pursuing.

It was obvious at the start that any superficial "tests" which we could make would be quite useless as a means to attain the end we had in mind. The processes of setting and hardening of cement are due essentially to chemical reactions but these may be very complicated. Cement contains a number of constituent compounds and each (presumably) of these reacts with water. In one type of reaction these compounds from a basic material, as hydrated lime, and other compounds less basic than was present before water was added. In another type of reaction water adds directly to the compounds. But there are other reactions to complicate the process. Some of the hydration or hydrolysis products may interact; the gypsum in the cement may react with some of the other components; the salts in the mixing water or in the water which later may come in

contact with the concrete, may bring about other changes. And finally, on exposure to air, the carbon dioxide may further alter the reaction products.

I have indicated that all of these chemical changes are brought about between the compounds in the cement on the one hand, and the water, dissolved salts, or air on the other hand. It is necessary to study these reactions if we are to learn the nature of the setting and hardening processes, and these processes must be well understood if we are to learn why they do not always proceed in the usual or desired manner. But we cannot study these reactions except we prepare the several compounds of cement in the pure form, and investigate the nature and rate of each reaction under ideal conditions. And finally we cannot prepare these pure compounds except we know what they are.

That, then, becomes our starting point: to find out just what portland cement consists of, the compounds which are in it, the limiting ratios of these constituents. We must know the influence of each material in the raw mixture upon the reactions occurring in the kiln, for each material exerts its special and characteristic influence in the production of the several product compounds.

So the problem oscillates from one phase to another, and the findings on any one part affect every other part of the study. A program as comprehensive as this cannot be covered in any brief period; nor can results of immediate application to the engineer to evolved in such a period. And of the results which will be obtained, while in some cases they may be applied in the field by the engineer, more often they must be applied in the manufacture.

Progress in Cement Research.—For two and a half years our staff in Washington has engaged in this study. This group of chemists, physicists and petrographers has, in that time, reported the virtual completion of about twenty separate problems, and eight papers have been published in the scientific and technical press. But in spite of this progress, the study has only begun. Many problems of technique have been worked out; many problems incidental to the major investigation have of necessity been attacked as their solution was essential before progress could be made.

This is not the place to go into a detailed account of our researches. For those I refer you to our several publications.[1] It will not be out of

[1] Paper No. 1, "Studies on the System CaO-Fe_2O_3-SiO_2," by W. C. Hansen and R. H. Bogue, *J. Am. Chem. Soc.*, *48* (1926), 1261.

Paper No. 2, "The Determination of Uncombined Lime in Portland Cement," by Wm. Lerch and R. H. Bogue, *Ind. Eng. Chem.*, *18* (1926), 739.

Paper No. 3, "A Digest of the Literature on the Constitution of Portland Cement Clinker," by R. H. Bogue, *Concrete*, July 1926 to February 1927.

Paper No. 4, "Portland Cement Research," by R. H. Bogue, *Proceedings*, A. S. T. M., *26* (1926).

Paper No. 5, "The Preparation and Optical Properties of Calcium Hydroxide Crystals," by F. W. Ashton and Raymond Wilson (in press).

Paper No. 6, "Studies on the System CaO-Al_2O_3-SiO_2. The Composition $8\,CaO + Al_2O_3 + 2\,SiO_2$," by W. C. Hansen, W. Dyckerhoff, F. W. Ashton, and R. H. Bogue (in press).

Paper No. 7, "The Preparation of Optically Clear Selenium for use in Index Media," by L. T. Brownmiller (in press).

Paper No. 8, "X-Ray Diffraction Measurements on Some of the Pure Compounds of Portland Cement," by E. A. Harrington (in press).

place, however, and may be of interest to you, for me to describe in a general way some of the studies that we are following and some of the interesting results that we are obtaining.

Of fundamental importance above all others, and for the reasons given above, are the studies which will give us information on the nature of the constituents in clinker. Some of these have been known for some time. Others have been guessed at, but the guesses of the various authorities do not always agree. One group believe that a complex compound containing lime, alumina and silica is present in clinker; another group that lime and one of the silicates form a loosely bound compound known as a solid solution; and still another group that the lime and silica form two separate compounds. Work in our laboratory led us to the latter belief, and this was corroborated by other authorities in this country. A recent co-operative study in our laboratory with a distinguished European scientist who formerly differed from us in this matter has been completed and is now in press. It is under joint authorship with him and bears testimony to the fruitfulness of co-operation.

But other components than lime and silica are present, and we must learn the manner of their combination. Into what compounds do the alumina, the iron oxide and the magnesia go? What change is observed in the composition of the product resulting from the use of different percentages of these and still other constituents of the raw material? These questions are being studied and two or three reports on this aspect of our work have been published.

It does not suffice, however, to learn only the influence of the several constituents on the *composition* of the product. In order that this information may be of significance to us, we must observe the corresponding influence on the *cement value* of the product. Different technique is required for these two lines of investigation. There are mechanical difficulties in the construction of a furnace which can be used to obtain data on constitution and at the same time yield a sufficient quantity of product to give us data on cement value. In a furnace which enables us accurately to determine the nature of the compounds formed, and the changes in these at varying composition of the raw mixture, we obtain temperatures up to 1625 deg. C. (2957 deg. F.) and control them to $\pm$ 1 deg., but the charge is only a fraction of a gram. A second furnace was built for obtaining a larger quantity of product so that tests could be made on strength and resistance to corrosion. In the latter furnace, temperatures up to 1550 deg. C. (2822 deg. F.) are obtained, controlled to $\pm$ 10 deg., and about 2 lb. of product are obtained per day. In both of these, contamination is completely prevented by the use of platinum containers for the charges and the employment of the electric current as the source of heat.

In these experiments, iron oxide is substituted systematically for alumina; magnesia or alkalies for lime; and so on with several other minor constituents. The effect of such substitutions is noted on the temperature of burning, on the completeness of combination, and on the strength under

compression of 1-in. cubes at various ages. A number of tests is added to give information on the setting and hardening and resistance to corrosion.

No ideal in manufacture can be considered entirely apart from the significance it bears to the production of concrete. I have referred to the numerous reactions of the setting and hardening processes. In order to learn more about these, several compounds which may occur in portland cement have been prepared in the pure form. These pure compounds have been made up into 1-in. cubes with sand and water, and submerged for various periods in water and in salt solutions. Reference may be found in the literature to a number of researches where concrete blocks have been made the object of somewhat similar studies. In these latter, the aggregate, the cement-water ratio and various additional materials have been varied to determine what technique or combination of materials gave the most resistant product. But in our tests, it is only the compound or mixture of compounds which changes. Hence we may observe not only the physical effect in each case, but the nature and the rate of the chemical reactions, the nature of the end-products and the effect of each reaction or end-product on the strength and appearance of each compound. Some of the compounds are found in this way to be resistant; some, non-resistant. The substitution of certain compounds for others or a change in the ratio of those already present has been found to alter profoundly the strength of a mortar or its resistance to certain types of corrosion.

To demonstrate the connection between the several studies, I may point to one instance. A certain substitution in a particular mixture with which we were working, was found to result in the formation of a new compound. The formation of this compound during the burning lowered the required temperature and made for more complete combination of the lime. Strength tests indicated a probable advantage as a result of the substitution, and finally this same substitution gave a product which seemed to be more resistant to the corrosive action of sulphate waters. Here then is a specific finding that looks most interesting. However, much more work is required before any definite application can be made.

Still another phase of our work which must be mentioned is the identification of the compounds which are produced in our furnaces, in commercial clinker, and in the reactions of setting and hardening. For this purpose the polarizing microscope is indispensable. Without it a correct interpretation of much of the work described would probably be impossible. Chemical analysis indicates ultimate composition but does not reveal the exact combinations of the elements or oxides that are present. These compounds may be determined, however, by a painstaking examination of the optical properties as determined under a microscope. In the case of new compounds these have first to be prepared in pure form, and their optical properties definitely ascertained. In no other manner that we know of could we as satisfactorily identify the compounds with which we are constantly working. In some very special cases, however, the microscope fails, and in these cases we are employing the X-ray diffraction

method of examination. This is a relatively new tool, but in some instances will give us desired information where the microscope will not do so, and in other cases may be used to corroborate important findings.

Summary and Conclusions.—I have given to you in this brief space some of our ideas on the study of portland cement in concrete engineering. I have indicated the nature of engineering research on the proportioning of the constituent materials that go to make up concrete, and have emphasized the remarkable achievements of the engineer towards the elimination of unsatisfactory concrete; toward the production of a concrete which is perfect in the great majority of cases.

But this age is one which looks ever to new conquests of achievement; to new levels of perfection. Progressive industry cannot stand still. It cannot rest on its laurels. It must advance. Nothing less than ultimate perfection can be our goal, but a 100 per cent efficiency under the standards of today may be far below the line under the standards of tomorrow. Our purpose then is to narrow the margin between our high water mark of efficiency and the 100 per cent line, and likewise ever to raise our ideas of perfection by the evolution of new and higher standards of value.

We are still in the preliminary stage of our research, but it is permissible to look ahead. We have been obliged to develop methods that would be comprehensive and adequate; we have had to build apparatus that would meet our extraordinary requirements; we have had to organize a new technique in highly specialized physico-chemical and optical measurements; we have had to meet unusual conditions by the discovery of causes, the invention of means for removing these causes, and the proper adaptation of those principles to our materials. Numerous incidental problems require solution before progress can be made on the major investigations. However, many of these latter are now well under way and the light we are receiving leads us to greater confidence in the reality of our goal.

But ultimate application of these researches can be made in safety and in confidence only after they have passed from the laboratory to the industrial plant, and in every case assurance must be had that any prospective change in manufacture shall be to the direct advantage of the final concrete. Such assurance cannot be obtained in a day or in a year. The element of time is very significant where the question of permanency is vital, as in concrete. But we may look forward to changes in portland cement which in one department or another will increase its usefulness. It may be found possible, for example, to alter the composition somewhat to give a product which, as concrete, will be more resistant to the action of corrosive waters, either acidic or alkaline or charged with such salts as sulphates or magnesium compounds. It may be found possible to improve the true early strength, or to diminish the volume change in setting or to improve plasticity for mortars, or to widen the field of usefulness. Any one of these would make the study very much worth while. The biggest mistake we could make, however, would be to rush through an idea that

had not been checked and confirmed and tested throughout all of the long range of influences which might be affected by its adoption.

A part of our program of research has been described and a few of our experimental results have been mentioned. Some of the difficulties and necessary refinements in our technique have been referred to, as well as some of the unusual equipment which has been constructed and employed. We are in the midst of a program of fundamental research on portland cement which is broader in scope than any that has ever before been undertaken. We are obtaining results which are both interesting and important. What ultimate effect these may have on concrete engineering is impossible to say. But it is certain that they will have some influence, and it is equally certain that this influence will be in the direction of an even more useful product—a more universally perfect concrete.

Coarse Aggregate in Concrete as a Field for Needed Research.

By Herbert J. Gilkey.*

Introduction.—Over a period of years several series of tests have been conducted in the concrete laboratory of the University of Colorado, with a view to comparing the strengths of mortars and concretes which were identical except for the absence or presence of coarse aggregate. Without exception the mortars gave higher strengths than the concretes. Usually these were from ten to twenty per cent greater, but the increases were sometimes as much as 50 per cent. In general the difference becomes less as the sizes of the particles, and the proportions of the coarse aggregate are diminished. The same facts have been noted by others and there are some published statements to this effect.

The usual point of view held by designers of concrete mixtures has been that the strength of concrete is essentially the strength of its mortar, the coarse aggregate being but an inert diluting material introduced mainly to save mortar. Some investigators have made statements to this effect, others imply it. The reasonableness of this point of view is evident to most persons. If tests do not verify it, the reason for the lack of agreement between the strength of the concrete and that of its mortar would appear to be traceable to one or more of the following factors:

1. The lack of homogeneity caused by the introduction of the coarse material.
2. Failure in bond between the mortar and the particles of coarse aggregate.
3. Weakness in the aggregate itself.
4. Chemical effects of the aggregate upon the mortar strength.

The last two reasons are, in general, absent. The usual definition of coarse aggregate, viz.: a material chemically inert, structurally sound, and at least as strong as the mortar matrix, excludes these.

Most investigations of concrete have been conducted with graded aggregate, generally one of the local products. Variations in size (or grading), shape, surfaces of aggregates, etc., may all exist as simultaneous variables. The effect of any one factor cannot be isolated, in such a case.

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Thought along these general lines has suggested to the writer the possibility and desirability of a fundamental and comprehensive investigation of coarse aggregates, with a view to running down each one of these factors in turn. The investigation planned is far beyond what it would be possible to conduct in any but a large laboratory with considerable resource. Regardless of whether or not such an investigation were actually launched, the writer is convinced that much benefit would accrue from getting these ideas before the leading present-day thinkers and investigators in concrete.

Just how the problem of isolating and attacking but one variable at a time, can best be approached can only be shown by a rather detailed outline of a hypothetical series of tests. The individual effects of shape of particle, size, surface, strength, and quantity are each to be run down in turn. Then would come some of the simpler combinations of factors, or elements, leading to general laws or conclusions that would be applicable to practical conditions.

The subject will be treated under three main headings, viz.:

1. Presentation of the problem.

2. Presentation of experimental evidence of a reconnaissance nature indicating probable trends and possible magnitudes.

3. Outline of the proposed investigation and discussion in some detail of possible methods of attacking the problem. This includes a consideration of:

(a) The selection of pertinent variables.

(b) Some of the difficulties that would be encountered and suggestions of ways to overcome or avoid them.

(c) General remarks, speculative or otherwise, that might have either a direct or indirect bearing upon the subject.

1. Presentation of the Problem.

(A) Some eminent authorities maintain that for workable mixtures, the strength of the concrete is the strength of its mortar.

(B) Some, while granting that the above may not be strictly true contend that the difference is so slight that it is not of practical importance.

(C) Others assert that the strength of the concrete is by no means the strength of its mortar and that the difference is of sufficient magnitude to make it far from negligible.

Those of the (A) persuasion deny the existence of a problem. Class (B) grant its existence but brand it not worth solving. It is something of academic interest from the standpoint of the theorist or pure scientist, perhaps, but one of those unprofitable speculative items of which the practical man should steer clear. To those of class (C) there seems to be a real problem of sufficient practical, as well as scientific, interest to challenge alike the curiosity and best efforts of the practical man and his less

practical, but intelligently curious, neighbor, the scientist or the pedagog who is perhaps more interested in reasons why than he is in dollars saved.

In class (A) we find the two foremost present-day investigators in the field of concrete, Talbot and Abrams.

The water-cement ratio strength criterion of Abrams excludes both the coarse and the fine aggregate from having any bearing upon strength of concrete except in so far as they influence the amount of water necessary for workability. It is presupposed, of course, that the aggregates, both fine and coarse meet the recognized requirements of aggregates and that proportions of cement, water and aggregates are within the recognized workable range. In other words Abrams, like Talbot, very properly excludes freak aggregates and freak proportions. Abrams' fineness modulus has nothing whatever to do with the strength or the water-cement ratio strength criterion. It is simply a constant obtainable from the sieve analysis, for the purpose of pre-determining the amount of water necessary to produce a concrete of satisfactory workability from given aggregates. Moreover, it is supposed to supply evidence as to whether or not the aggregates are so graded as to produce concrete not too harsh for proper workability or so fine as to require too much water for proper economy of cement. If the fineness modulus were not known in advance, the information supplied by it could be obtained by trial mixtures. In so far as either fineness modulus or surface modulus (which is simply another constant based on the sieve analysis, exactly analogous to fineness modulus as regards scope and function) are imperfect as criterions, the actual mixture will need experimental modification.

Thus, although the fineness modulus is an incidental and more or less useful adjunct of the water-cement ratio theory, the water-cement ratio is the sole strength criterion proposed by Abrams and, within the range of proper workability, strength will vary with this ratio and with it alone. Size, quantity and grading of aggregates may be shifted at will, within the workability range, without influencing the strength of the concrete. This is the doctrine of Abrams and leaves no loop hole for any difference in strength between workable mortars and concretes of equal water cement ratios. Nor does it leave any loop hole for differences in strength due to grading as long as the same water cement ratio is maintained and workability is not sacrificed.

With Abrams theory the difference in strength by virtue of grading arises from the fact that the coarser the grading, without over-shooting into the realms of harsh unworkable mixtures, the greater the apparent wetness of mix a given water ratio will provide and it is possible to get a workable mixture with a lower water ratio than can be attained with smaller sized aggregates (lower fineness or higher surface moduli). At constant water cement ratio these experiments, and those of others seem to indicate very definitely that the larger the grading of the coarse aggregate, the weaker the concrete. If the water ratio be also variable then we have a concrete stronger because of less water, as size of aggregate in-

creases, and weaker by virtue of larger aggregate (possibly because of less surface in contact with the mortar). Thus there are two opposing factors simultaneously present and there may be some doubt as to which way the balance might be thrown in a particular case. This point is, however, one to be considered later. For the present the purpose is to show clearly that Professor Abrams' position is inevitably that of Class A. "Mortars and concretes of equal water cement ratios have equal strengths and the present problem does not therefore exist."

Professor Talbot expounder of the admirably fundamental voids-cement ratio theory is also very definitely of the Class (A) persuasion. Quite aside from the present point at issue, it is urged that those, interested in a basic understanding of the concrete problem, read the "General Discussion on Qualities of Aggregates," p. 71, Bulletin No. 137, University of Illinois, by Talbot and Richart. The whole point of view set forth by the voids-cement ratio theory, using as it does absolute volumes as the basis from which to work, is very enlightening.

The mere fact that developments may show that certain important points of either the voids-cement ratio theory or the water-cement ratio theory, are not supported, does not lessen the value of either theory in its other applications. Nor does it detract one iota from the very splendid work that these investigators have performed in advancing the cause of better and more intelligently designed concrete mixtures.

As to the evidence that Talbot is of Class (A) mind, it will possibly be best to quote his own words as used on pages 80 and 81 of Bulletin 137:

> "The points representing the mortars were somewhat more scattered than the points in diagrams representing the tests of concrete, and they generally fell below the curve of the concrete tests. Two other sets of mortar specimens, 6 x 12-in. cylinders, made in connection with later series of tests of concretes, gave compressive strengths at or above the curve for the relation between voids-cement ratio and strength of concrete. Several series of tests made elsewhere indicate that the voids-cement-strength relation holds for both mortars and concretes. Differences between the strength of mortars and concretes as indicated by the voids-cement relation may be expected to be due to differences in relative water content, in manipulation in making the specimen (such as those affecting compactness), and in conditions attendant on differences in size of specimen.
>
> "Other things being the same, it may be asserted that below a reasonable limit for the amount of the coarse aggregate, the strength of the concrete is identical with the strength of the mortar composing it. 'Other things' refer to relative water content, manipulation in placing, and compactness of mortar and concrete, absorptive properties of coarse aggregate, manner of curing and testing, etc. In making a comparison of the relative strengths of mortars and concretes, special care should be taken to secure identical conditions. It is further assumed

that the strength of the particles of coarse aggregate is at least as great as the strength of the mortar and that the adhesion of cement to the coarse aggregate is as great as to the fine aggregate. Under this view, the coarse aggregate becomes only a filler, concentrating the mortar and making it richer for a given amount of cement in a fixed volume of concrete. This conception of the function of the mortar simplifies the process of designing the mixture, for the voids-cement ratio of the concrete in good workable mixtures will be essentially the same as that of the contained mortar. This conception also leads to a better comprehension of the source of the strength and the effect of variations in mixtures."

The "two other sets of mortar specimens, 6 x 12-in. cylinders, made in connection with later series of tests of concretes," that "gave compressive strengths at or above the curve for the relation between voids-cement ratio and strength of concrete," were the series that first set the writer to seriously considering this problem.

The indication given by these series was probably more nearly the correct indication than that from the "Several series of tests made elsewhere" that "indicate that the voids-cement-strength relation holds for both mortars and concretes." In all of the several series that have been run (only a few of which are published in connection with this paper) every precaution has been taken to avoid the "differences in manipulation, relative water content" and other factors enumerated. In the two Illinois series mentioned, every precaution of this kind was also taken.

There has been no evidence in any series to support the belief that the strengths of similar mortars and concretes are equal. Moreover these two series were from 10 to 20 per cent stronger than several concrete series with identical mortars. These identical concrete series were made from well graded approved aggregates such as sound gravels, crushed limestones, granites, etc. The 10 to 20 per cent excess strength of the mortar series of these Illinois tests accords very perfectly with the usual differential found in the many series of Colorado tests.

In considering these comparisons of mortars and concretes one more point should not be overlooked. In all of these laboratory tests, both at Illinois and at Colorado the aggregates were dry as used. No extra water was added in the concretes to allow for absorption by the coarse aggregate. The aggregates used were sound and had low absorptions, 1.5 to 2 per cent by weight. Nevertheless this was enough to extract some water from the mortar of the concrete and therefore lower the water-cement ratio of the concrete below that of the mortar specimens. Thus this factor gave the concrete an advantage over the mortar. Nevertheless the concrete was decisively weaker.

That more recent developments at Illinois may be modifying the very definite position taken on pp. 80 and 81 of Bulletin 137, is perhaps indicated by the utterance of Professor Richart *Proceedings*, American Concrete Institute, 1925, pp. 177-178 quoted below:

"Mr. Hutchinson's observations regarding the effect of the size of the coarse aggregate are substantiated to some degree by some tests conducted recently at the University of Illinois. A series of tests of the compressive strength of concrete mixtures taken from the table of proportions given in the report of the Joint Committee on Specifications for Concrete and Reinforced Concrete includes concretes made with three gradations of sand and three of coarse aggregate, three consistencies and three designed strengths. The coarse aggregates used were gravels of gradations designated as No. 4 to ¾ in., No. 4 to 2 in., and ¾ to 2 in. The principal feature of the tests was that the concrete made with the first coarse aggregate gave unexpectedly high strengths, and that containing the last, the ¾ to 2 in. material, gave relatively low strengths. That is, this rather uniformly graded coarse aggregate, in which the particles were of large size and which had a high fineness modulus, gave low strengths, while of the better graded aggregates, the one having the smaller maximum size gave the best results. These unexpected variations in strength are not only variations from the designed strength, but also variations to a marked degree from the curves expressing the relation between the strength and water-cement ratio and voids-cement ratio.

"It would seem that the explanation might lie in the extent and condition of the surface of the coarse aggregates. While all of the aggregates were from the same source, the surface moduli, in the order mentioned above were 1.27, 0.92, and 0.35, so that the coarse aggregate giving the least strength had only about one-fourth as much surface area as the one giving the highest strength. The strength of the concrete may also be affected by the extent of the bearing contact and interlocking between particles of coarse aggregate, in a way similar to that observed in the action of railway track ballast."

In this discussion, concretes and mortars are not being compared, but to concede that identical concretes except for differences of gradations, of course aggregates, can have different strengths is as much of a reversal of the previous stand as to admit that mortars and concretes of identical voids-cement ratios have not identical strengths.

All of the discussions on pp. 177 to 179 (*Proceedings* A. C. 1, 1925) are very pertinent to this subject, as are also many portions of the paper just preceding, by Mr. G. W. Hutchinson. Several of the opposing points of view are well set forth.

Mr. Hutchinson here, and elsewhere[1] has produced rather convincing proof that the compressive strength of the concrete, and especially its resistance to impact, decrease appreciably with increase in size of coarse aggregate particles. This is directly contrary to Abrams' contention but is very well supported by the evidence of these tests as regards compressive strength. This series does not include any impact tests.

[1] "Concrete," Sept., 1926, pp. 13-16, Figs. 2, 3, 5, and 6.

Definite support of the Class (C) contention is given by George M. Williams.[2] In Hool and Kinne "Reinforced-Concrete and Masonry Structures," foot of p. 17, he states:

"An assumption sometimes made is that the strength of the concrete will be that of the mortar used. Such an assumption is erroneous since the concrete strength may be as little as 50 per cent of that of the mortar."

Again at the foot of p. 28, same text, he states:

"Owing to the limited capacities of available testing machines, it has been customary on some work, where aggregates up to 4 or 6 in. maximum size have been used, to mold small specimens with the fine material screened from the concrete. Such tests will no doubt indicate comparative qualities but may give little idea of the actual strength values obtained in the work as these mortars may test from 50 to 100 per cent higher than the original concrete as placed."

The range of percentage difference between mortar and concrete strengths given by Professor Williams is higher than most of the test data of the present investigations show. Some of the comparative data of Fig. 7 do show relative strengths of about the percentages mentioned.

In the light of the foregoing, it would appear that the question as to whether or not coarse aggregate, of accepted kinds has an important bearing upon strength, aside from its influence upon water requirement or voids-cement ratio, is at least debatable.

2. *The Presentation of the Experimental Evidence.*

In handling this phase of the subject, the intent has been to make both Tables and Figures so self explanatory that they could be digested with little or no text explanation.

There are nevertheless certain important points that might pass unnoticed without some comment.

Probably little need be said regarding the tables as their main features generally appear more visibly on the curve sheets. With one point in view, however, it is desired that the tables be subjected to rather rigorous scrutiny. It is sometimes true that very satisfying and regular trends are obtained from averaging large numbers of results that are among themselves erratic and unsatisfactory. Sometimes it is felt that a needed slogan is: "Not how many, but how well."

In these tests it is believed that, with very few exceptions, the individual specimen tells the same story as does its group average. To avoid confusion from extraneous matter, certain points should be clearly kept in mind. The points at issue come to a focus in the following proposition.

"The strength of the concrete is (or is not) the strength of its mortar."

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[2] Professor of Civil Engineering, University of Saskatchewan, Saskatoon, Saskatchewan, Canada.

To test the truth or falsity of the proposition, let us suppose that we have a large vat of mortar. Some of this we make into mortar specimens. Some of it we mix with approved coarse aggregate and call concrete. We vary the kind of aggregate, its amount, and the sizes of its particles. If our mortar has not been altered in its properties by the aggregate filler, it would appear that the equal or unequal test strengths from these mortar and concrete specimens would demonstrate the truth or falsity of the proposition.

It is easy enough to cure and test the specimens, but just how are we to be assured that differences in strength do not merely indicate that the properties of the mortar were altered? Perhaps the aggregate extracted water from the mortar by absorption or perhaps the concrete was tamped or worked in a different manner from the mortar specimens. Any of the several factors mentioned by Talbot may be present. This is especially true for the usual case in which the large vat of mortar is but figurative. The mortar specimens are usually made up from independently weighed and mixed batches.

This is where the fundamental methods of analysis made possible by the voids-cement theory of Talbot come to the rescue. Identical mortars will have equal V/C ratios and the voids of the mortar, V_m, will be equal regardless of whether that mortar be located between particles of coarse aggregate or be in a specimen by itself. Just how closely the mortar of the mortar specimens has been duplicated in the concrete will therefore appear in the comparative values of V_m, and V/C. All the other properties give indications as to extent of uniformity in similar specimens.

The evidence of most of these tests, individually and collectively, indicate that the concretes did contain identical mortars and that we are justified in assuming that the strength test results mean just what they appear to mean.

So far as the writer can see there remains only one possible prop for the objector who is still reluctant to concede that the strength of the concrete and the strength of the mortar are not truly equal. He may say that these concretes are nearly all of one-size aggregates and that with proper grading from small to large, the strength of the concrete will be increased to that of its mortar. Such an argument is in direct opposition to the original contention. It is a distinct concession to admit that any grading of aggregate whatever, as long as the mortar voids, or voids-cement ratio, or water-cement ratio remains unaltered, can effect the strength of the concrete. By either of these theories, the sole gain to be obtained from grading the aggregates is:

(a) To save in quantity of mortar used and thus obtain an equal strength concrete at lower cost or,

(b) To obtain a mixture that can be made wet enough for workability with less water, thereby reducing the water voids and therefore the voids-cement ratio, or the water-cement ratio, as the case may be.

This is not a quibble. It is perfectly proper that any basic theory be expected to be consistent with itself. It is only by such seemingly hair-splitting distinctions, perhaps, that incompleteness can be detected and causes sought.

Regardless of whether or not it is a consistent thing for the exponent of one of the theories in question to assert that a better grading of the coarse aggregate would produce concrete with the strength of its mortar, the point is well taken.

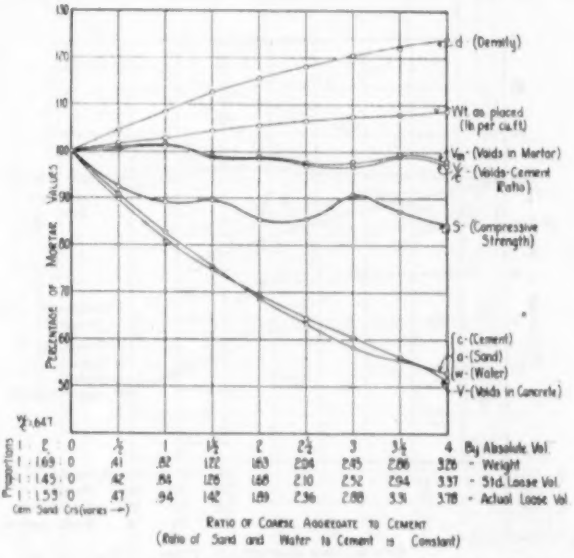

FIG. 1.—QUANTITY OF COARSE AGGREGATE VARIED. EFFECT UPON VARIOUS PROPERTIES OF THE MORTAR AND CONCRETES.

Series—Spring 1925; Specimens 6 x 12 in.; Tested at 28 days after standard curing; Batches accurately weighed out in the equivalent of absolute volume proportions; Fine aggregate—Washed Boulder Creek sand (0-4); Coarse aggregate—White sandstone (1-1¼); Consult Tables 1, 2 and 3 for further information; Also Notation, Symbols, and Definitions; See Fig. 3 for another graph of same series.

The writer has endeavored to settle this point by running several series with graded coarse aggregates. He has not been able to find any support whatever for the contention, as long as proper precautions were taken to insure the constancy of the mortar, as regards the water-cement, or the voids-cement ratios.

The only series of those here recorded, that had a graded coarse aggregate is that of Table 8 and Fig. 8. The coarse aggregate in that series was ⅜ to 1. No indication of strengths differing from those of the one size aggregate series is apparent. In much of his investigational work, the writer uses one size aggregates in order to limit the number of simul-

taneous variables as far as possible. The present graded series was incidental rather than intentional. It was found that there was not enough material of one size for the proposed series, so several lots were thoroughly premixed for use in the series.

Four different series of tests with graded aggregates of both gravel and crushed stone were run several years ago. They showed the mortar as having the greatest strength, and a strength variation with different amounts of coarse aggregate more or less proportional to the proportion of mortar in the mixture. The same findings were obtained when the

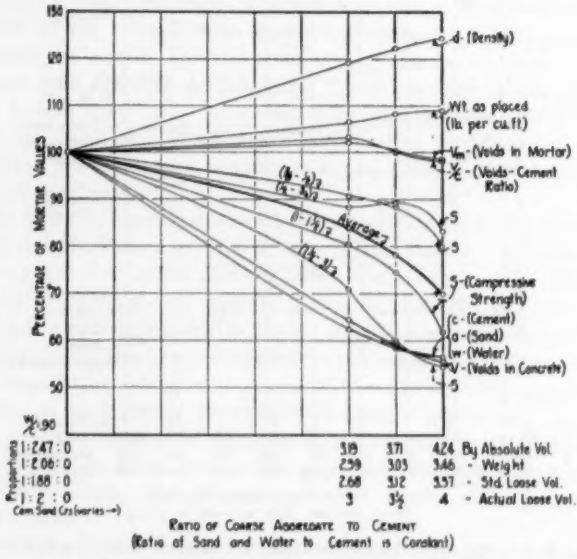

FIG. 2.—QUANTITIES OF COARSE AGGREGATE VARIED FOR FOUR SIZES OF AGGREGATE. MORTAR CONSTANT.

Series—Spring 1926; Specimens—6 x 12 in.; Fine aggregate—Boulder Creek sand (0-4); Coarse aggregate—White sandstone (quartzite); Standard curing; 28-day tests; See Tables 1, 2 and 4; Figs. 4 and 6 are other graphs of these same data. See also Notation, Definitions, and Analytical Relations.

mortar was extracted from the concrete (or the rocks extracted from the mortar) by wet screening. This was accomplished rapidly with little chance for moisture loss in screening. The results showed the mortar stronger as usual.

The two mortar series at the University of Illinois, mentioned earlier, showed their extra strength in competition with graded orthodox aggregates, as was previously mentioned.

Lastly the evidence is invariably that of mortar strengths falling above the water-cement and voids-cement-strength curves, rather than the concretes of one size grading being below the curves. (See Fig. 7).

Fig. 1 shows a somewhat irregular, but nevertheless pronounced, falling off in compressive strength with increase in amount of coarse aggregate. This is in spite of the facts that the voids-cement and water-cement ratios are slightly less for the concretes than for the mortars, due probably to some absorption of water by the dry coarse aggregate. No attempt was made to correct the mixing water for this absorption. None of these mixtures contained enough coarse aggregate to be harsh or unworkable. Had

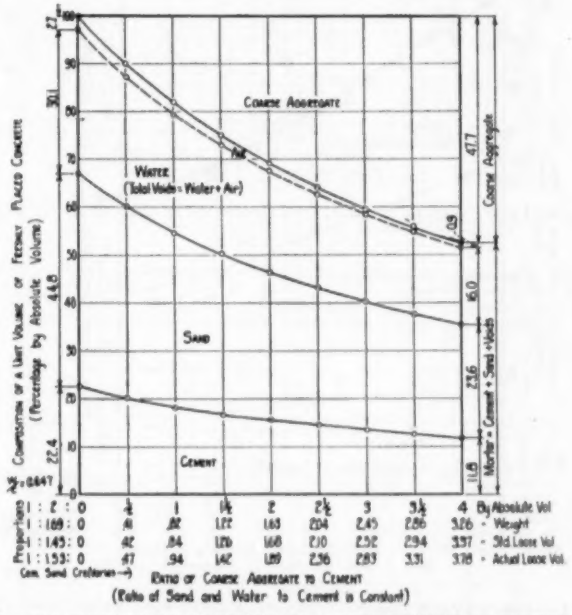

FIG. 3.—PERCENTAGE COMPOSITION OF A UNIT VOLUME OF FRESHLY PLACED CONCRETE CONTAINING VARYING AMOUNTS OF COARSE AGGREGATE. MORTAR CONSTANT FOR ALL SPECIMENS.

Series—Spring 1925; Specimens—6 x 12 in. See Tables 1, 2, and 3 and Notation, Symbols, and Definitions. Fig. 1 contains additional information on same series. Fine aggregate—Boulder Creek sand (0-4); Coarse aggregate—White sandstone (1-1¼).

this been the case there would have been an increase in voids as the right hand side was approached.

What Fig. 1 shows for a dry mixture (water-cement ratio of 0.647) Fig. 2 shows for a much wetter mixture (water-cement ratio of 0.90) and for four different sizes of coarse aggregates.

In Fig. 1 (Table 3) with an aggregate 1: 1¼ the concrete at the extreme right developed 84 per cent of the strength of the mortar, while in Fig. 2 (Table 4) the smallest aggregate, ⅜ to ½, developed about 80 and the largest, 1½: 3, about 54 per cent of the mortar strength. The

average for the four sizes of aggregate in Fig. 2, (Table 4) is 70 per cent. In neither series did any concrete approach the strength of the mortar.

Figs. 3 and 4 (Table 3 and 4) constitute a slight digression, as neither shows anything that pertains to relative strengths. They do, however, show just what a unit volume of two rather extreme grades of concrete (dry and wet), of a range extending to mortar only at the left,

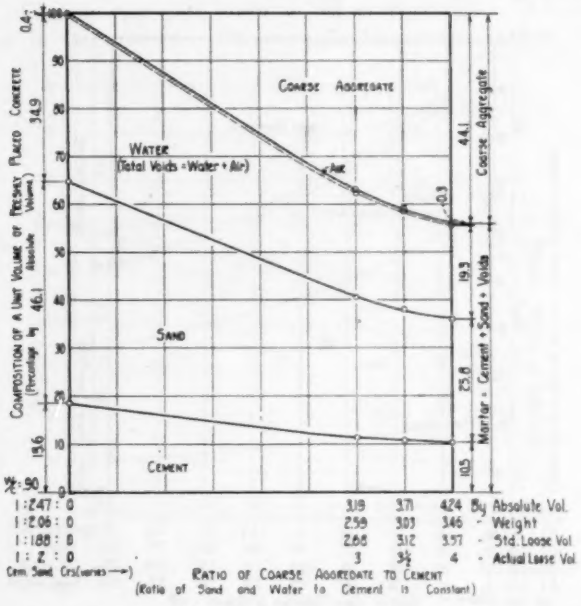

FIG. 4.—PERCENTAGE COMPOSITION OF A UNIT VOLUME OF FRESHLY PLACED CONCRETE CONTAINING VARYING AMOUNTS OF COARSE AGGREGATE. MORTAR CONSTANT.

Series—Spring 1926; Specimens—6 x 12 in. See Tables 1, 2 and 4: Figs. 2 and 6 are other graphs of these same data. See also Notation, Definitions, and Analytical Relations; Fine aggregate—Boulder Creek sand (0-4); Coarse aggregate—White sandstone (quartzite); Standard curing.

contain. Few concrete minded people think in such terms as are here graphically shown. All should.

For example a cubic yard of mortar requires about twice as much cement as a yard of usual concrete from the same mortar. The diagram shows this at a glance. Used in conjunction with the strength curves of Figs. 1 and 2 for these same concretes, many points of relative economy, strength, etc., may be almost solved at a glance. Perhaps the most valuable of all points to be gained from such a study is the correct conception of the meaning of such terms as voids, densities, and absolute

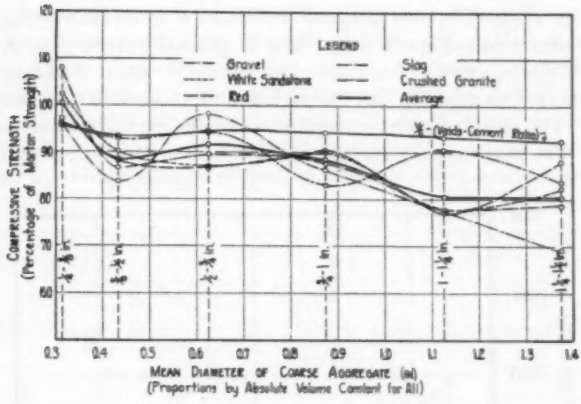

FIG. 5.—EFFECT OF SIZE FOR FIVE DIFFERENT COARSE AGGREGATES. MORTARS CONSTANT.

Series—Winter 1924-25; Specimen—6 x 12 in. Standard curing; 28-day tests. w/c = 0.647. Proportions—1 : 2 : 0, 1 : 2 : 3, absolute volume; Fine aggregate—Boulder Creek sand (0-4); Coarse aggregate—Variable; See also Tables 1, 2, 5 and 6 and Notation, Definitions, and Analytical Relations.

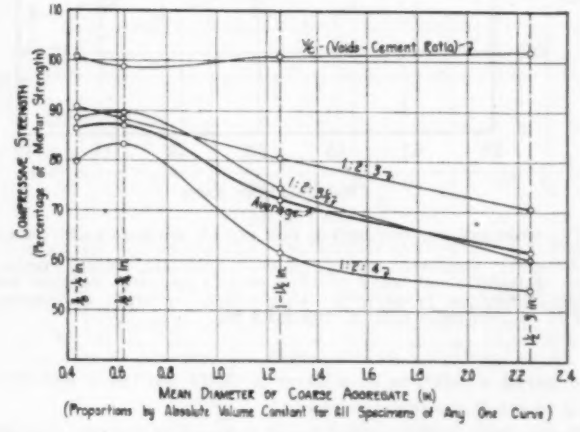
Compressive Strength (Percentage of Mortar Strength)
Mean Diameter of Coarse Aggregate (in)
(Proportions by Absolute Volume Constant for All Specimens of Any One Curve)

FIG. 6.—EFFECT OF SIZE FOR THREE DIFFERENT PROPORTIONS OF COARSE AGGREGATE. MORTAR CONSTANT.

Series—Spring 1926; Specimens—6 x 12 in.; Standard curing; 28-day tests; Fine aggregate—Boulder Creek sand (0-4); Coarse Aggregate—White sandstone (quartzite); See Tables 1, 2 and 4; Same series is also plotted on Figs. 2 and 4; See also Notation, Definitions, and Analytical Relations.

volumes as applied to mortars and concretes. It is interesting to note that in concrete mixed much drier than is generally used (Fig. 3, $W/C = 0.647$) the mixing water constitutes nine-tenths or more of the total voids in the concrete as placed. Air voids are almost zero in the concretes of practice. Yet most people's conception of voids in concrete is the honeycombing that may appear on the surface as cavities or caves from the

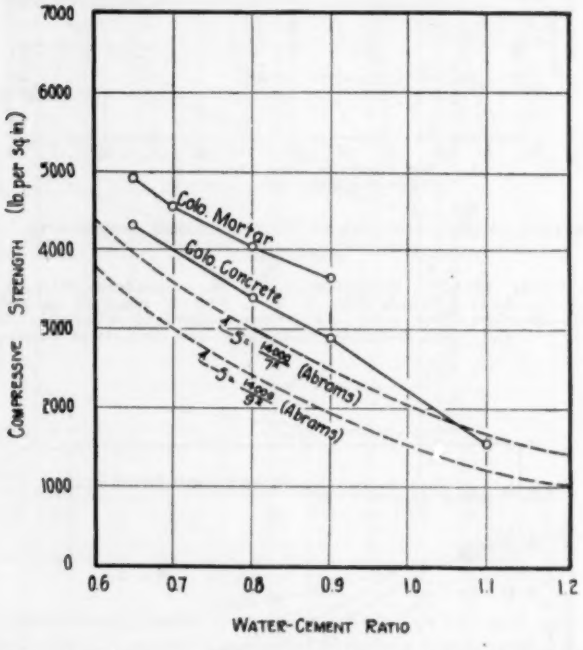

FIG. 7.—MORTARS VS. CONCRETES FOR EQUAL WATER-CEMENT RATIOS.

Series—Several; Standard curing; w/c—(variable); 28-day tests; Fine aggregate—Boulder Creek sand (0-4); Coarse aggregate—Several, but usually White sandstone (quartzite); Proportions—several; Specimens—Concrete 6 x 12 in., Mortar 6 x 12 in. and 2 x 4 in.

use of too harsh a mixture or a layer a little too thick for the spading or tamping to reach the lower part of it.

Very unsightly honeycombing, may increase the voids scarcely a perceptible amount and have a negligible effect upon strength. It is the closely spaced oft recurring minute invisible pore spaces, spreading the particles of cement and sand apart, ever so slightly, that really raise the voids percentage and weaken the concrete or the mortar. A cubic foot of concrete occupies 1,628 cu. in. of space. About 20 per cent of this space is voids, i. e., some 325 cu. in. Three or 4 cu. in. of honeycombing

next to the mold is most unsightly but certainly does not take on any considerable importance in comparison to the total voids content of the concrete. If the block were without coarse aggregate, i. e., were entirely of mortar, it would contain between 500 and 600 cu. in. of voids space. To illustrate, a bag of portland cement (1 cu. ft.) weighs 94 lb. Its specific gravity is about 3.15, i. e., the solid weight of the material if it contained no voids would be about 196.8 lb. In other words a bag of port-

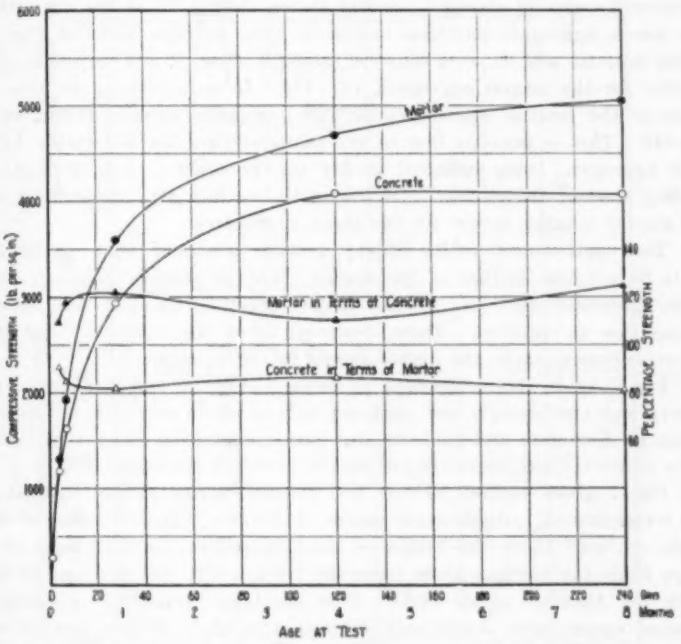

FIG. 8.—MORTARS VS. CONCRETES. STRENGTH EFFECT OF AGE FOR STANDARD CURING.

Series—Spring 1926; Specimens 6 x 12 in.; Fine aggregate—Boulder Creek sand (0-4); Coarse aggregate—Crushed red sandstone (¾-1); See Tables 1, 2 and 8; w/c = 0.90; Proportions—1 : 2 : 4 actual loose volume: Proportioning done by accurate weighing but loose weight per cu. ft. was used as basis for weights of materials.

land cement is more than half air, yet the cement does not seem porous to the eye. In a box of large spheres many cavities are visible, yet the total ratio of voids space in the box of spheres is considerably less than that in the bag of cement. In the one case there are many small solids interspersed with small cavities, both practically invisible to the eye. In the other case there are large volumes of solid matter and less large volumes of space, both visible.

This digression is entirely apart from the main purpose of the paper and absurdly elemental for many. Nevertheless it may open up for some few a new avenue of thought that might lead to a more intelligent understanding of concrete in one phase or another. Without further apology, it is therefore offered as being in the interests of better concrete which after all is what explains and justifies our presence here.

In Fig. 5 (Tables 5 and 6) there is illustrated by the series of overlapping wavy curves for five widely different coarse aggregates the same downward trend in strength, as was shown on Fig. 2, as the diameter of the coarse aggregate particles increased. The average curve of Fig. 5 is fairly uniform and shows a concrete strength about 80 per cent that of the mortar for the largest aggregate, 1¼ : 1½. It is interesting to note that some of the smaller aggregates produce strengths exceeding that of the mortar. This is possibly due to the relatively greater absorption by the finer aggregate, being sufficient to dry up the concrete to a strength exceeding that of the mortar. Of course the surface area for contact with the mortar is much larger for the small aggregates.

The voids-cement ratios of the average concretes seem to be from 92 to 96 per cent of that of the mortar. This is entirely contrary to the close agreement that has existed in the other series. At this time no explanation is ventured. These, however, favor the concrete. The lower the voids-cement ratio, the higher should be the strength.

Fig. 6 again shows the data of Table 4, Figs. 2 and 4, showing very clearly and convincingly the apparent falling off in strength for both increase in diameters and quantities of coarse aggregate. The voids-cement ratio of mortar and concretes are here in excellent agreement.

Fig. 7 gives various mortar and concrete series plotted against the two water-cement ratio-strength curves of Abrams. The concretes of these series are well above the higher of the Abrams' curves over most of the range while the mortars show strengths from 125 to 147 per cent of those given by Abrams' upper curve. The carefully controlled concretes of Abrams' upper curve would only average from 68 to 82 per cent of what the mortars of these series have given. The mortars of our series average about 110 to 125 per cent of the strengths of the concretes or the concretes show strengths of about 80 or 90 per cent of the mortars. It is doubtless fairer to compare the Colorado mortars with Colorado concretes than to use the generalized Abrams' curves.

There is no figure for the data of Table 7. The two groups of specimens listed (of *M* and *C* numbering) are controls made by different operators on different days in connection with various other series. The group of twenty-three specimens for which only the average results are given were made under quite similar conditions and were fully as consistent with one another as the two groups that are fully given. Under these conditions, it would seem that the very consistent relation between mortar and concrete strengths extending over the relatively large numbers of similar specimens must be given fair weight as supporting evidence to

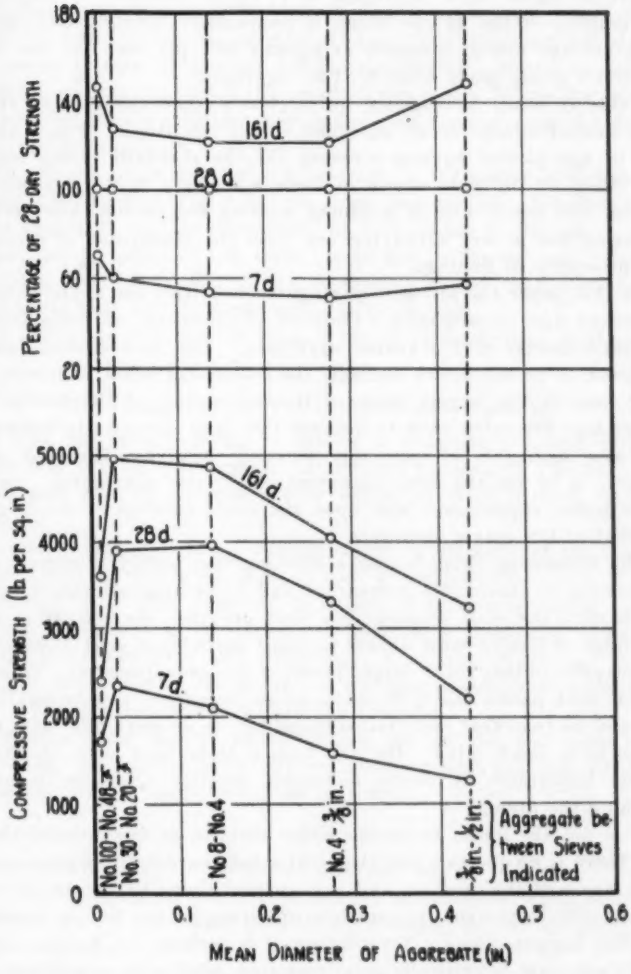

FIG. 9.—EFFECT OF SIZE OF PARTICLE UPON THE STRENGTH OF MORTAR SPECIMENS AT VARYING AGES.

Made Summer 1925; Specimens 2x4 in.; w/c = 70; Aggregate—Washed natural sand of one size; Proportions, 1 : 2.50 : 0 by weight, 1 : 2.96 : 0 by absolute volume, 1 : 2.15 : 0 by std. loose volume, 1 : 2.26 : 0 by actual loose volume; Standard moist curing for all. For properties of aggregate, see table 2. Ages at test: 7, 28, and 161 days.

that already considered. In further support of the indications noted, the concrete having a coarse aggregate of 1: 1¼-in. material developed, from an averaging of the 23 specimens, a compressive strength 84.5 per cent of the average mortar strength as against 89.5 per cent for the average of the (C) group made with ¾: 1-in. aggregate.

Table 8 seems to indicate quite clearly that the greater strength of the mortar exists for all ages and curing conditions. Fig. 8 shows a curve of age plotted against strength for the standard curing condition only. Many of the cases shown in Table 8 are very lightly touched. But wherever the touch came it produced nothing but corroborative evidence. The series was a very satisfying one from the standpoint of consistency and uniformity of findings.

In this paper the problem was approached from the angle of most of the present day investigators. Concrete is considered as being composed of a sand mortar and a coarse aggregate. The sand mortar has been considered to be the active element, the coarse aggregate a passive filler. Theory and design largely centered themselves around the mortar. The findings here presented seem to indicate that it is incorrect to assume that the coarse aggregate is even approximately passive. The sand or fine aggregate is by far the more important of the two aggregates. Its effect on the water requirement and upon the voids relation is much greater than that of the coarse aggregate.

The interesting thing is that a dividing line, purely arbitrary, should have become so thoroughly entrenched and be as unquestioningly accepted as if it were the basic scientific law that underlies the cementing action.

Grains of cement must adhere to small particles of sand in exactly the same manner as they do to larger pieces of the same material. To classify a pebble that passes the ¼-in. mesh as an important mortar-making material and its twin that just "failed to get by" as an inert filler does appear strange to a small mind. Hit the twin a light blow with the hammer and his importance is vastly increased, possibly after the manner of "smashing the atom."

It is not the intent to become either childish or facetious on the subject. There is no question but that real minds have devoted genuine study to this phase of the question and that current usage has behind it enough of profundity to prevent its complete upsetting by the first or second upstart that happens along. Nevertheless, it is perhaps not amiss to suggest that we not cease to critically scrutinize such arbitrarily established "dead lines" in the light of each new development.

Fig. 10 represents one small and perhaps superficial reconnaissance venture into this field. The results are at least worthy of passing mention. With a constant water-cement ratio of 0.70 specimens were made from one size sands from the same source using one proportion for all, viz., 1 to 2.5 by weight. The finest sand 100 to 48 grading gave too dry a mixture and the 30 to 20 was rather dry but shows the maximum strength at two ages out of the three ages of test. After passing the peak the strength

falls off rapidly as the size of particle increases. The curves are exactly analogous to the concrete curves of Fig. 2, 5, and 6. It would appear that any comprehensive investigation of coarse aggregates would sooner or later, by mere analogy have to be extended to a consideration of many of the same factors as applied to fine aggregates. Moreover, there is little question that any trend that might exist would ignore the, possibly convenient, but purely arbitrary lines established for purposes of definition, simpler manipulation, or what not.

The present paper has set out to treat of the coarse aggregate and it is not intended that Fig. 9 intrude upon that project. Whether or not the fine aggregate may be considered, in its relation to the coarse aggregate, as simply, "more of the same," need not bother us now. There is before us a virgin field and it is a restriction rather than an extension of boundaries that may be found much to be desired.

We now come to a consideration of the third subdivision of the paper, viz.:

3. The Outline of the Proposed Investigation.

It does not seem feasible to outline a complete proposed series of tests as was originally intended. A hurried perusal of the field, with more or less random comments may suffice to start desired trains of thought.

If coarse aggregate has a bearing upon the strength of the concrete, that effect may be due in whole or in part to the following:

1. Quantity, 2. size, 3. shape, 4. nature of surface including texture, treatment, and cleanness, 5. strength of particle, 6. durability of particle, 7. expansibility (swelling and shrinking), 8. specific gravity or relative weight as compared with the mortar, 9. stiffness as compared with mortar matrix, and 10, arrangement within the concrete.

Doubtless other factors should appear on the list and some of the items overlap one another badly. Nevertheless a certain amount of thoughtful attention may be suggestive if not instructive.

1. Quantity.

The quantity might be effective in several ways.

(a) More aggregate per unit of cement subjects the latter to heavier duty and it seems reasonable that strength should fall off as quantity of aggregate increases.

(b) If the coarse aggregate be a weakening (or a strengthening) element for any one or more of the other causes enumerated, then increasing it would enhance the effect.

(c) As the coarse aggregate is increased the concrete becomes more heterogeneous and stresses are likely to be more seriously localized within the mass producing wedging, splitting or other non-uniform and unfavorable stress effects. This overlaps the scope of point No. 10 (arrangement within the mass).

2. Size.

The experimental evidence considered seems to indicate that size of particle is an important factor. It has often been considered desirable to crowd just as much large material into the mass as is compatible with its dimensions. From the standpoint of economy of cement there is no question of the advantage of doing this in mass work. Abrams has always urged that the coarser the grading the stronger the concrete (within the workable limit). As previously noted there is some justification for this because of the slightly lower water-cement ratios that can be attained with coarser mixtures.

These experiments, and the quotations from Professor Williams[3] would indicate an exactly opposite point of view, as being possibly more nearly correct. A large rock has relatively much less surface to be gripped by the mortar. There is a possibility and a probability that initial compressive failure within the mass of the concrete is a failure in bond between mortar and aggregate. Break a large stone in half and without changing the volume of rock you have added two new surfaces for contact with mortar. Below some limiting diameter of particle all diameters might be sufficiently well gripped to prevent the bond or slippage failure. The curves of Fig. 9 did not so indicate however, as the diameter of particle seemed to bear the same relation to strength for sizes well into the realm of the fine aggregates. The fact that gravel concretes of good sound washed particles often seem to be somewhat weaker than similar concretes from crushed stone is, if true, further evidence of possible weakness in bond. The crushed stone is interlocked and dovetailed into the mixture in a far more thorough manner than are the gravel particles. This suggests the next factor.

3. Shape of particle.

There is no question but that thin or elongated particles of materials, otherwise suitable, are a source of weakness in concrete. The particles are well embedded doubtless but bear alternately against the soft mortar bedment and the harder aggregate particles. They are thus loaded like beams within the mass and earlier breakdown will occur because of flexural failure of aggregates within the concrete mass. The genuineness of this condition was illustrated a few years ago. Two coarse aggregates were made by breaking up ¼ and ⅛-in. scraps of plate glass until they would pass the 1¼-in. sieve. The concretes from these aggregates failed at lower load than concretes of more compactly shaped aggregates. The thinner plate glass carried much less load than the ¼-in. The most interesting feature, however, was the distinct groaning or grinding sound from within the mass that could be heard for some time prior to reaching the ultimate compressive load. There was little question but that the sound was merely

[3] Hool and Kinne, "Reinforced Concrete and Masonry Structures," foot of pages 17 and 28.

the grinding together of fractured ends that resulted from the beam action of cross-bearing fragments. In this connection attention may be called to one common error of "practical concrete men." They break open a mass of concrete and if the fracture passes through some of the particles of aggregate, instead of passing around them, they conclude that the mortar was stronger than the aggregate and that about the maximum obtainable strength had been reached. The phenomenon, of course, means only that the particle was rather well anchored each way from the plane of fracture and therefore failed by bending or shearing across. A mortar as strong as any of the usual concrete aggregates is very rare indeed. Gravels rarely fracture across any but elongated or rotten particles because of the shallow bedding either way from the plane of failure.

4. Nature of the surface, etc.

This factor in certain of its aspects has been covered in one way or another by specifications but there are probably few of the elements about which less is really known. Rough, smooth, and polished surfaces; both clean and coated with films of clay, paraffine, etc., should be experimentally studied. Absorbent and non-absorbent surfaces and aggregates should be compared. Metallic and mineral surfaces, galvanized and etched surfaces, ones knobbed or studded, and those corroded with rust, etc., should be considered. An almost infinite range of possibilities exists but a careful survey would probably reveal ways to group certain types in a manner that would greatly abbreviate the field to be covered.

5. Strength of particle.

Some aggregates, notably cinders, are used in which the aggregate is admittedly weaker than its mortar matrix. In cases in which such aggregates have been used, the chemical factors, shape, and possibly other complications have mitigated against any very satisfying evidence as to the extent of the effects of the different influencing factors. Coke, burned shale, etc., are other light weight low strength aggregates that have been used. Locally there is another, of interest. Much of the gravel in parts of Colorado contains a high percentage of decomposed granite, schist, and hornblende fragments that are washed round and appear to be as sound as any trap, basaltic or quartz gravel. When specimens from this aggregate are broken, many fractures pass through the softer fragments that might easily have gone around them, had any great resistance been offered. Aggregates of predetermined strength should be investigated. Entire lots much weaker than the mortar, of equal strengths, and stronger should be used.

6.—Durability of particles.

Just what decomposition can normally take place within the body of concrete for various conditions of use is an important subject that has

only been approached from a few angles as yet. It is of special interest in connection with marine structures, swamp or alkali problems, and fire protection.

7. Expansibility—swelling and shrinkage.

This is closely allied to 5 and 6 and although it has received some attention and needs more, the writer has no new suggestions to offer.

8. Specific Gravity or relative weight of aggregate as compared with the mortar.

Heavy aggregates tend to settle to the bottom of the forms and light ones rise. Within the usual range this may not be serious but it is a factor in the use of heavy basaltic rock or precious metal slag (S. G. of 3.53), if the mixture contains less aggregate than is required for saturation. Concretes made from iron balls as coarse aggregate were composed of iron ball concrete in the lower portion and mortar in the upper. In investigating the surfaces of aggregates, some cases of using metal balls, etc., would doubtless be desirable to cover certain limiting cases such as (a) the highly polished surface; (b) the absolutely strong and unyielding aggregate; (c) that truly non-absorbent, etc. This could easily be done where the mixture was crowded full of the aggregate. It would not be feasible for mixtures of lesser aggregate contents unless perhaps aluminum (S. G. 2.7) or equivalent light weight materials were employed.

9. Stiffness.

There is no question about the undesirable elements of stress distribution that are bound to exist within a composite body of materials differing greatly in their stiffness, rigidity, or moduli of elasticity. Synthetic aggregates varying in stiffness from that of the mortar, each way over quite a range would appear to be desirable in investigating this phase.

10. Arrangement within the concrete.

Concrete in compression is sometimes likened to masonry construction. The analogy is a poor one for the haphazard arrangement of particles introduces shear, bond stress, wedging and tension which may be present only within the more or less homogeneous separate masonry units. A vertical column of aggregate particles may form a stiff column for a considerable distance within the member. Such a pillar would carry more than its share of the load as long as the condition continued to exist. A readjustment would probably be the initial failure of the member. This question should be studied by using orderly and disorderly arrangements of spheres, cubes, and pyramids within the mass. Wedging should be both avoided and sought, aggregate should be unevenly distributed over the compressive cross-section, giving the equivalent of eccentric loading through lack of homogeneity in the concrete. Much more might be said and many more suggestions offered.

RECOMMENDED SPECIAL PROCEDURE.

The problem doubtless appears to be quite appalling. It is. To obtain aggregates for testing out any one factor without having simultaneously present other variable factors such as shape, size, surface condition, specific gravity or weight complications, etc., is obviously impossible within the range of engineering materials.

But the condition does not appear to be entirely hopeless. All factors could be controlled by the use of synthetic aggregates of any desired richness of mix, shape, size, surface treatment, etc. Moreover the weight can be controlled very perfectly. It would appear that properly designed and made synthetic aggregates for basic investigations in the proposed field of aggregates offers a splendid starting point for one of the most needed and fundamental investigations offered by the whole field of concrete research.

We have been using concrete for many years, thanks to the venturesome practical man who was willing to employ a material of which he and others knew something less than nothing. Is it not time that we who are endowed with more curiosity than action, bestir ourselves to the point of trying to learn something about this very useful structural element?

ACKNOWLEDGMENTS.

The writer is very grateful to Prof. C. L. Eckel, head of the department of civil engineering, University of Colorado, for the encouragement that he has at all times offered in the small research projects undertaken. Mr. Wm. L. Eager, senior civil engineering student, has labored hard and long in working up the data of the tests. Prof. F. G. Allen, head of the department of engineering drawing, made distinct sacrifices of time and labor to render valuable assistance. It is to Prof. Whitney C. Huntington, head of the department of civil engineering, at the University of Illinois, that the writer is largely indebted for the idea of a synthetic coarse aggregate as offering a possible rational approach to the problem. That and other points were brought out in the course of a casual conversation some two years or so ago.

CONCLUSIONS.

1. Very eminent authorities differ as to whether or not the coarse aggregate bears any practical relationship to the strength of the concrete.

2. These tests in common with others here and elsewhere seem to indicate beyond reasonable doubt that the coarse aggregate is closely linked to strength. The concrete is not as strong as its mortar.

3. There therefore seems to be defined a real need for a thorough and fundamental investigation of the different aspects of coarse aggregate in concrete. This could only be undertaken by some large well-equipped laboratory or several such. Even then the problem offers what seem to be insurmountable difficulties in eliminating simultaneous variables unless synthetic precast coarse aggregates of mortar be used.

4. It is felt that it would be a splendid thing for the cause of concrete, if such an investigation, as is here outlined, could be adequately and properly launched. In the absence of such a launching it is still felt that to get these problems before the eyes and minds of the thinking concrete public cannot help but be a splendid thing for the cause of concrete.

NOTATION, DEFINITIONS AND ANALYTICAL RELATIONS.

S = compressive strength in lb. per sq. in. (Standard curing, tested at 28-days, specimens 6 x 12 in. cylinders, if not otherwise stated.)

a = absolute volume of fine aggregate in a unit of volume of freshly-placed concrete;

b = absolute volume of coarse aggregate in a unit of volume of freshly-placed concrete;

c = absolute volume of cement in a unit of volume of freshly-placed concrete;

d = density or solidity ratio of the freshly-placed concrete;

v_m = voids (air and water) in a unit of volume of the mortar mixture of cement, fine aggregate, and water as it exists in the concrete;

v = voids in a unit of volume of concrete. This, of course, will be equal to $1-d$.

It is evident that

$$a + b + c = d = 1 - v \quad \ldots\ldots (1)$$

Since the mortar and the coarse aggregate together make up the unit of volume, it is also evident that

$$\frac{c + a}{1 - v_m} + b = 1 \quad \ldots\ldots (2)$$

A third equation derived from equations (1) and (2) will be found useful:

$$v = v_m\ (1-b) \text{ or } b = 1 - \frac{v}{v_m} \quad \ldots\ldots (3)$$

The notations used accord with Bulletin 137 University of Illinois—Talbot and Richart (p. 16).

TABLE 1—OUTLINE OF TESTS—INDEX AND SUMMARY

Series	Description	Tables No.	Fig. No.	Proportions		W/C
1	2	3	4	5	6	7
1924–25 Winter	Sizes and kinds of Coarse Aggregate. (Spec. 6 x 12 in.).	2, 5, 6	5	1:2:0 1:2:3	Absolute Volume	0.647
1925 Spring	Quantity of Coarse Aggregate. (Spec. 6 x 12 in.).	2, 3	1, 3	1:2:0 to 1:2:4	Absolute Volume	0.647
1925 Summer	Size of Fine Aggregate (Mortar only). (Spec. 2 x 4 in.).	2	9	1:2.5:0	By weight	0.70
1925 Summer Also others	Water-Cement Ratio. Concrete vs. Mortar strengths. (Spec. 2 x 4 in.). (Spec. 6 x 12 in.).	2	7	1:2:3 1:2.5:0 1:2:3 1:2:0	Actual Loose Weight. Absolute Volume Absolute Volume.	Variable
1926 Spring	Sizes and Quantities of Coarse Aggregates. (Spec. 6 x 12 in.).	2, 4	2, 4, 6	1:2:0 1:2:3 1:2:3.5 1:2:4	Actual Loose Volume	0.90
1926 Spring	Curing Conditions. Mortar vs. Concrete. (Spec. 6 x 12 in.).	2, 8	8	1:2:3	Actual Loose Volume	0.90

TABLE 2—PROPERTIES OF THE MATERIALS

Kind	S. G.	Weight per cu. ft.			A.S.T.M. Standard		Absorption, Per Cent by weight
		Solid	Actual Loose	Standard Loose A.S.T.M. C29–21	Density	Voids	
1	2	3	4	5	6	7	8
Cement (U. S. brand).....	3.15	196.8	.....	94.0	0.478	0.522	....
Sand (prior to 1926).....	2.66	166.0	104.0	109.5	0.660	0.340	1.30
Sand (Spring, 1926).....	2.63	164.1	103.0	97.0	0.591	0.409	1.30
White Sandstone.....	2.57	160.5	81.4	91.0	0.568	0.432	0.91
Red Sandstone.....	2.50	156.0	79.0	87.0	0.560	0.440	1.50±
Slag.....	3.53	220.5	.....	120.4	0.546	0.454	0.0+
Crushed Granite.....	2.61	163.0	.....	88.0	0.540	0.460	1.40±
Gravel.....	2.58	161.0	130.0	98.3	0.610	0.390	1.30

TABLE 3—VARIATION IN THE QUANTITY OF COARSE AGGREGATE: MORTAR CONSTANT

Made: Spring, 1925 *Specimens:* 6 x 12 in. $W/C = 0.647$ for all. *Age at Test:* 28-days. *Fine Aggregate:* Boulder Creek Sand (0–4). *Coarse Aggregate:* White Crushed Sandstone (quartzite) (1–1¼). For further data see Tables 1 and 2, and Figs. 1 and 3. See also Notation, Definitions, and Analytical Relations.

Spec. No.	*S*	*V/C*	Unit Wt.	*e*	*a*	*b*	*d*	*V*	*Vm*	*w*
1	2	3	4	5	6	7	8	9	10	11
1:2:0 MORTAR (Proportions by Absolute Volume)										
76M	5920	1.455	137.8	0.225	0.449	none	0.674	0.326	0.326	0.305
96M	5130	1.484	136.7	0.223	0.446	none	0.669	0.331	0.331	0.302
116M	5670	1.488	136.4	0.223	0.445	none	0.668	0.332	0.332	0.302
Average	5573	1.478	137.0	0.224	0.447	none	0.670	0.330	0.330	0.303
Per cent Mortar	(100)	(100)	(100)	(100)	(100)	none	(100)	(100)	(100)	(100)
1:2:½ CONCRETE										
71	5260	1.500	138.9	0.200	0.401	0.099	0.700	0.300	0.333	0.271
91	4770	1.483	139.3	0.201	0.401	0.100	0.702	0.298	0.331	0.272
111	5420	1.483	139.3	0.201	0.401	0.100	0.702	0.298	0.331	0.272
Average	5150	1.489	139.2	0.201	0.401	0.100	0.701	0 299	0.332	0.272
Per Cent Mortar	(92.4)	(100.8)	(101.6)	(89.7)	(89.7)	.....	(104.6)	(90.6)	(100.6)	(89.8)
1:2:1 CONCRETE										
72	4910	1.505	140.5	0.181	0.363	0.182	0.726	0.274	0.336	0.245
92	4800	1.519	140.3	0.181	0.363	0.181	0.725	0.275	0.336	0.245
112	5200	1.400	141.1	0.182	0.365	0.182	0.729	0.271	0.331	0.247
Average	4970	1.501	140.6	0.181	0.364	0.182	0.727	0.273	0.334	0.246
Per Cent Mortar	(89.2)	(101.8)	(102.7)	(80.8)	(81.4)	. ...	(108.5)	(82.7)	(101.2)	(81.2)
1:2:1½ CONCRETE										
73	4780	1.405	142.6	0.167	0.334	0.251	0.752	0.248	0.331	0.226
93	5050	1.472	143.1	0.167	0.335	0.252	0.754	0.246	0.329	0.226
113	5170	1.420	144.0	0.169	0.337	0.254	0.760	0.240	0.322	0.229
Average	5000	1.459	143.2	0.168	0.335	0.252	0.755	0.248	0.327	0.227
Per Cent Mortar	(89.7)	(98.8)	(104.5)	(75.0)	(74.9)	.....	(112.7)	(75.2)	(99.1)	(74.9)
1:2:2 CONCRETE										
75	4600	1.506	143.5	0.154	0.307	0.307	0.768	0.232	0.335	0.209
95	4970	1.438	145.2	0.155	0.311	0.311	0.777	0.223	0.324	0.210
115	4660	1.423	145.3	0.156	0.311	0.311	0.778	0.222	0.322	0.211
Average	4743	1.456	144.7	0.155	0.310	0.310	0.774	0.226	0.327	0.210
Per Cent Mortar	(85.1)	(98.5)	(105.6)	(69.2)	(69.4)	.....	(115.5)	(68.5)	(99.1)	(69.3)
1:2:2½ CONCRETE										
77	5030	1.489	144.9	0.143	0.286	0.357	0.787	0.213	0.331	0.193
97	3720[1]	1.412	146.3	0.145	0.289	0.361	0.785	0.205	0.310	0.197
117	4500	1.443	145.8	0.144	0.288	0.360	0.792	0.208	0.325	0.195
Average	4765	1.448	145.7	0 144	0.288	0.359	0.791	0 209	0.322	0.195
Per Cent Mortar	(85.6)	(97.0)	(106.3)	(64.3)	(64.4)	.....	(118.0)	(63.3)	(97.6)	(64.4)

[1] Omitted from average.

TABLE 3—VARIATION IN THE QUANTITY OF COARSE AGGREGATE: MORTAR CONSTANT (*Continued*)

Spec. No.	*S*	*V/C*	Unit Wt.	*c*	*a*	*b*	*d*	*V*	*Vm*	*w*
1	2	3	4	5	6	7	8	9	10	11
1:2:3 CONCRETE										
78	4680	1.462	146.7	0.134	0.268	0.402	0.804	0.196	0.328	0.182
98	5340	1.415	147.6	0.135	0.269	0.405	0.809	0.191	0.321	0.183
118	5120	1.414	147.6	0.135	0.269	0.405	0.809	0.191	0.321	0.183
Average	5047	1.430	147.3	0.135	0.269	0.404	0.807	0.192	0.323	0.183
Per Cent Mortar	(90.6)	(96.7)	(107.5)	(60.2)	(60.2)	.....	(120.4)	(58.2)	(97.9)	(60.4)
1:2:3½ CONCRETE										
79	4670	1.540	146.5	0.124	0.248	0.437	0.809	0.191	0.339	0.168
99	4820	1.420	148.6	0.127	0.252	0.442	0.821	0.179	0.321	0.172
119	5050	1.420	148.4	0.126	0.252	0.442	0.820	0.180	0.323	0.171
Average	4847	1.463	147.8	0.126	0.251	0.440	0.817	0.183	0.328	0.171
Per Cent Mortar	(87.0)	(99.0)	(107.8)	(56.2)	(56.2)	.....	(121.9)	(55.5)	(99.4)	(56.4)
1:2:4 CONCRETE										
80	4470	1.458	148.6	0.118	0.236	0.474	0.828	0.172	0.327	0.160
100	5170	1.440	148.9	0.118	0.237	0.475	0.830	0.170	0.324	0.160
120	4430	1.394	149.7	0.119	0.238	0.477	0.834	0.166	0.318	0.161
Average	4690	1.432	149.1	0.118	0.237	0.475	0.831	0.169	0.323	0.160
Per Cent Mortar	(84.2)	(96.0)	(108.8)	(52.7)	(53.0)	.....	(124.0)	(51.2)	(97.9)	(52.8)

TABLE 4—VARIATION IN QUANTITY AND SIZE OF COARSE AGGREGATE: MORTAR CONSTANT

Made: Spring, 1926. *Specimens:* 6 x 12 in. *W/C* = 0.90 for all. *Age at Test:* 28 days. *Fine Aggregate:* Boulder Creek Sand 0-4. *Coarse Aggregate:* White Crushed Sandstone (grading variable). For further data see Tables 1 and 2, and Figs. 2 and 4. See also Notation, Definitions, and Analytical Relations.

Spec. No.	*S*	*V/C*	Unit Wt.	*c*	*a*	*b*	*d*	*V*	*Vm*	*W*
1	2	3	4	5	6	7	8	9	10	11
1:2:0 MORTAR (Proportions by Actual Loose Volume)										
7	3988	1.903	133.9	0.186	0.460	none	0.646	0.354	0.354	0.351
15	3640	1.851	135.6	0.188	0.466	none	0.654	0.346	0.346	0.346
27	3770	1.934	133.2	0.185	0.457	none	0.642	0.358	0.358	0.349
Average	3799	1.895	134.2	0.186	0.461	none	0.647	0.353	0.353	0.349
Per Cent Mortar	(100)	(100)	(100)	(100)	(100)	none	(100)	(100)	(100)	(100)
1:2:3 CONCRETE (Size ¾-½)										
22	3385	1.931	143.6	0.116	0.288	0.372	0.776	0.224	0.357	0.219
30	3510	1.897	143.9	0.117	0.289	0.371	0.778	0.222	0.353	0.220
Average	3448	1.914	143.8	0.117	0.288	0.372	0.777	0.223	0.355	0.219
Per Cent Mortar	(90.8)	(101.0)	(107.2)	(62.9)	(62.5)	.....	(119.7)	(63.2)	(100.6)	(62.8)

TABLE 4—VARIATION IN QUANTITY AND SIZE OF COARSE AGGREGATE: MORTAR CONSTANT (*Continued*)

Spec. No.	*S*	*V/C*	Unit Wt.	*c*	*a*	*b*	*d*	*V*	*Vm*	*W*
1	2	3	4	5	6	7	8	9	10	11
1:2:3 CONCRETE (Size ¼-¾)										
2	3386	2.078	141.2	0.114	0.283	0.366	0.763	0.237	0.374	0.215
10	3325	1.931	143.6	0.116	0.288	0.372	0.776	0.224	0.357	0.219
Average	3356	2.005	142.4	0.115	0.286	0.369	0.769	0.231	0.366	0.217
Per Cent Mortar	(88.4)	(105.7)	(106.2)	(61.8)	(62.0)	.....	(118.5)	(65.4)	(103.7)	(62.2)
1:2:3 CONCRETE (Size 1-1½)										
21	3007	1.948	143.2	0.116	0.287	0.371	0.774	0.226	0.360	0.219
29	3110	1.872	144.5	0.117	0.290	0.374	0.781	0.219	0.350	0.220
Average	3059	1.910	143.9	0.116	0.289	0.373	0.777	0.223	0.358	0.220
Per Cent Mortar	(80.5)	(100.8)	(107.2)	(62.4)	(62.7)	.....	(119.7)	(63.2)	(101.4)	(63.1)
1:2:3 CONCRETE (Size 1½-3)										
1	2518	2.043	141.6	0.115	0.284	0.366	0.765	0.235	0.371	0.217
9	2870	1.965	142.8	0.116	0.286	0.370	0.772	0.228	0.362	0.219
Average	2694	2.004	142.2	0.115	0.285	0.368	0.769	0.231	0.367	0.218
Per Cent Mortar	(70.9)	(105.7)	(106.0)	(61.8)	(61.8)	.....	(118.5)	(65.4)	(104.0)	(62.5)
1:2:3.5 CONCRETE (Size ¾-½)										
24	3360	1.927	144.5	0.110	0.271	0.407	0.788	0.212	0.358	0.207
32	3360	1.927	144.7	0.110	0.271	0.407	0.788	0.212	0.358	0.207
Average	3360	1.927	144.6	0.110	0.271	0.407	0.788	0.212	0.358	0.207
Per Cent Mortar	(88.5)	(101.7)	(107.8)	(59.1)	(58.8)	.....	(121.5)	(60.1)	(101.4)	(59.3)
1:2:3.5 CONCRETE (Size ¼-¾)										
4	3520	1.900	145.2	0.110	0.272	0.409	0.791	0.209	0.354	0.207
12	3290	1.785	147.2	0.111	0.276	0.415	0.802	0.198	0.338	0.198
Average	3405	1.841	146.2	0.111	0.274	0.412	0.797	0.203	0.346	0.203
Per Cent Mortar	(89.6)	(97.1)	(109.0)	(59.7)	(59.4)	.....	(122.8)	(57.5)	(98.0)	(58.2)
1:2:3.5 CONCRETE (Size 1-1½)										
23	2888	1.900	145.2	0.110	0.272	0.409	0.791	0.209	0.354	0.207
31	2775	1.908	145.0	0.110	0.272	0.408	0.790	0.210	0.355	0.207
Average	2831	1.904	145.1	0.110	0.272	0.408	0.791	0.209	0.355	0.207
Per Cent Mortar	(74.6)	(100.4)	(108.1)	(59.1)	(59.0)	.....	(121.8)	(59.2)	(100.6)	(59.3)
1:2:3.5 CONCRETE (Size 1½-3)										
3	2004	2.037	143.2	0.108	0.268	0.404	0.780	0.220	0.369	0.204
11	1956	1.783	147.2	0.111	0.276	0.415	0.802	0.198	0.339	0.198
Average	2280	1.910	145.7	0.109	0.272	0.410	0.791	0.209	0.354	0.201
Per Cent Mortar	(60.0)	(100.8)	(108.6)	(58.6)	(59.0)	...	(121.8)	(59.2)	(100.3)	(57.6)

TABLE 4—VARIATION IN QUANTITY AND SIZE OF COARSE AGGREGATE: MORTAR CONSTANT (*Continued*)

Spec. No.	S	V/C	Unit Wt.	c	a	b	d	V	Vm	w
1	2	3	4	5	6	7	8	9	10	11
1:2:4 CONCRETE (Size ¾-½)										
26	3130	1.962	144.6	0.104	0.255	0.437	0.796	0.204	0.363	0.196
34	2930	1.838	146.6	0.105	0.258	0.444	0.807	0.193	0.347	0.193
Average	3030	1.900	145.6	0.104	0.256	0.441	0.801	0.199	0.355	0.195
Per Cent Mortar	(79.8)	(100.3)	(108.5)	(55.9)	(55.5)	.....	(123.4)	(56.4)	(100.6)	(55.9)
1:2:4 CONCRETE (Size ½-¾)										
6	3451	1.800	147.3	0.105	0.260	0.446	0.811	0.189	0.342	0.189
14	2861	1.692	148.7	0.107	0.262	0.450	0.819	0.181	0.329	0.181
Average	3156	1.746	148.0	0.106	0.261	0.448	0.815	0.185	0.335	0.185
Per Cent Mortar	(83.1)	(92.1)	(110.3)	(57.0)	(56.6)	.....	(125.6)	(52.4)	(94.9)	(53.0)
1:2:4 CONCRETE (Size 1-1½)										
25	2330	1.903	145.7	0.104	0.257	0.441	0.802	0.198	0.354	0.195
33	2340	1.961	144.6	0.104	0.255	0.437	0.796	0.204	0.363	0.196
Average	2335	1.932	145.1	0.104	0.256	0.439	0.799	0.201	0.358	0.196
Per Cent Mortar	(61.5)	(101.8)	(108.1)	(55.9)	(55.5)	.....	(123.5	(56.9)	(101.4)	(56.2)
1:2:4 CONCRETE (Size 1½-3)										
5	1985	1.866	146.0	0.105	0.257	0.442	0.804	0.196	0.351	0.196
13	2148	1.903	145.7	0.104	0.257	0.441	0.802	0.198	0.348	0.196
Average	2067	1.885	145.8	0.104	0.257	0.442	0.803	0.197	0.350	0.196
Per Cent Mortar	(54.4)	(94.5)	(108.7)	(55.9)	(55.7)	.....	(129.2)	(55.8)	(99.1)	(56.2)

CONCRETE AVERAGES (all sizes of coarse aggregate)

Proportions	S	V/C	Unit Wt.	c	a	b	d	V	Vm	W
1	2	3	4	5	6	7	8	9	10	11
ACTUAL VALUES										
1:2:3	3186	1.958	142.9	0.116	0.287	0.370	0.773	0.227	0.361	0.219
1:2:3.5	2969	1.896	145.4	0.110	0.272	0.410	0.792	0.208	0.354	0.205
1:2:4	2647	1.866	146.1	0.105	0.258	0.441	0.805	0.196	0.349	0.193
Average	2934	1.907	144.8	0.110	0.272	0.407	0.790	0.210	0.355	0.206
PERCENTAGE VALUES (in terms of mortar)										
1:2:3	83.9	103.6	106.5	62.4	62.3	.....	119.0	64.3	102.2	62.9
1:2:3.5	78.2	100.0	108.3	59.1	59.0	.....	122.4	58.9	100.3	58.7
1:2:4	69.7	98.4	108.9	56.5	56.0	.....	124.4	55.5	98.9	55.3
Average	77.0	100.6	107.9	59.2	59.0	.....	122.0	59.5	100.4	58.9

TABLE 4—VARIATION IN QUANTITY AND SIZE OF COARSE AGGREGATE: MORTAR CONSTANT (*Continued*)

CONCRETE AVERAGES (all proportions of coarse aggregate)

Size	*S*	*V/C*	Unit Wt.	*c*	*a*	*b*	*d*	*V*	*Vm*	*W*
1	2	3	4	5	6	7	8	9	10	11
ACTUAL VALUES										
¾-1½	3279	1.913	144.7	0.110	0.272	0.407	0.789	0.211	0.355	0.207
½-¾	3306	1.864	145.5	0.111	0.274	0.410	0.794	0.206	0.349	0.202
1-1½	2742	1.915	144.7	0.110	0.272	0.407	0.789	0.211	0.357	0.208
1½-3	2347	1.933	144.6	0.109	0.271	0.407	0.788	0.212	0.357	0.205
Average	2919	1.906	144.9	0.110	0.272	0.408	0.790	0.210	0.355	0.206
PERCENTAGE VALUES (in terms of mortar)										
¾-1½	86.3	100.8	107.8	59.2	59.0	.....	121.8	59.7	100.5	59.3
½-¾	87.0	98.4	108.4	59.6	59.5	.....	122.7	58.3	98.8	57.9
1-1½	72.2	101.0	107.8	59.2	59.0	.....	121.8	59.7	101.1	59.6
1½-3	61.8	102.0	107.8	58.6	58.7	.....	121.7	60.0	101.1	58.7
Average	77.0	100.6	107.9	59.2	59.0	.....	122.0	59.5	100.4	58.9

TABLE 5—VARIATION IN SIZE OF COARSE AGGREGATE: CONCRETES FROM FIVE AGGREGATES; MORTARS CONSTANT

Made: Winter, 1924–25. *Specimens:* 6 x 12 in. $W/C = 0.647$ for all. *Age at Test:* 28 days. *Proportions:* Mortar, 1:2:0; Concrete, 1:2:3 by absolute volume. *Fine Aggregate:* Boulder Creek Sand (0–4). *Coarse Aggregate:* Varies (grading and kind). For further data see Tables 1, 2, and 6, and Fig. 5. Same material as Table 6, but differently arranged. See also Notation, Definitions, and Analytical Relations.

NOTE.—In Column 1. White = White sandstone (quartzite). Red = Red sandstone (almost quartzite). Slag = Heavy precious metal slag. Cr. Gr. = Crushed Granite (rather rotten). Grav. = Boulder Creek Gravel.

Coarse Aggregate, Kind	*S*	*V/C*	Unit Wt.	*c*	*a*	*b*	*d*	*V*	*Vm*	*W*
1	2	3	4	5	6	7	8	9	10	11
1:2:0 MORTAR (Proportions by Absolute Volume)										
None	4480	1.503	136.1	0.222	0.444	none	0.666	0.334	0.334	0.301
None	4390	1.464	137.3	0.224	0.448	none	0.672	0.328	0.328	0.304
None	4930	1.493	136.4	0.223	0.445	none	0.668	0.333	0.333	0.302
Average	4600	1.487	136.6	0.223	0.446	none	0.669	0.332	0.332	0.302
Per Cent Mortar	(100)	(100)	(100)	(100)	(100)	none	(100)	(100)	(100)	(100
1:2:3 CONCRETE (Size ¼-¾)										
White	4710 (102.4)	1.483	146.1	0.133	0.267	0.400	0.800	0.200	0.333	0.180
Red	4630 (100.7)	1.442	145.2	0.134	0.269	0.403	0.806	0.194	0.325	0.182
Slag	4970 (108.0)	1.358	173.3	0.136	0.272	0.408	0.816	0.184	0.311	0.184
Crushed Granite	4400 (95.7)	1.448	148.1	0.134	0.269	0.403	0.806	0.194	0.325	0.182
Gravel	4470 (97.2)	1.416	147.9	0.135	0.270	0.404	0.809	0.191	0.321	0.183
Average	4636	1.429	152.1	0.134	0.269	0.404	0.807	0.193	0.323	0.182
Per Cent Mortar	(100.7)	(96.1)	(111.3)	(60.1)	(60.3)	.....	(120.6)	(58.1)	(97.3)	(60.4)

TABLE 5—VARIATION IN SIZE OF COARSE AGGREGATE: CONCRETES FROM FIVE AGGREGATES. MORTARS CONSTANT (*Continued*)

Coarse Aggregate, Kind	*S*	*V/C*	Unit Wt.	*c*	*a*	*b*	*d*	*V*	*Vm*	*W*
1	2	3	4	5	6	7	8	9	10	11
1:2:3 CONCRETE (Size ⅜–½)										
White	4140 (90.0)	1.423	147.4	0.135	0.269	0.404	0.808	0.192	0.322	0.183
Red	4050 (88.0)	1.408	145.8	0.135	0.270	0.405	0.810	0.190	0.319	0.183
Slag	3990 (86.7)	1.362	173.3	0.136	0.271	0.408	0.815	0.185	0.313	0.184
Crushed Granite	4270 (92.8)	1.371	149.5	0.136	0.271	0.407	0.814	0.186	0.314	0.184
Gravel	3890 (83.9)	1.336	149.4	0.136	0.273	0.409	0.818	0.182	0.308	0.184
Average	4068	1.380	153.1	0.136	0.271	0.406	0.813	0.187	0.315	0.184
Per Cent Mortar	(88.4)	(92.8)	(112.1)	(61.0)	(60.8)	.....	(121.5)	(56.3)	(94.9)	(61.1)
1:2:3 CONCRETE (Size ¼–¾)										
White	4010 (87.2)	1.422	147.7	0.135	0.270	0.404	0.809	0.191	0.321	0.183
Red	4000 (86.9)	1.389	146.3	0.135	0.271	0.406	0.812	0.188	0.317	0.183
Slag	4120 (89.6)	1.390	172.6	0.135	0.271	0.406	0.812	0.188	0.317	0.183
Crushed Granite	4350 (94.6)	1.353	150.0	0.135	0.273	0.409	0.817	0.183	0.310	0.183
Gravel	4520 (98.3)	1.436	147.5	0.134	0.270	0.403	0.807	0.193	0.323	0.182
Average	4200	1.398	152.8	0.135	0.271	0.406	0.811	0.189	0.318	0.182
Per Cent Mortar	(91.9)	(94.1)	(111.9)	(60.5)	(60.8)	.....	(121.2)	(56.9)	(95.8)	(60.4)
1:2:3 CONCRETE (Size ¾–1)										
White	4140 (90.0)	1.444	147.0	0.134	0.269	0.403	0.806	0.194	0.325	0.182
Red	4170 (90.6)	1.348	147.2	0.136	0.272	0.408	0.816	0.184	0.311	0.184
Slag	4070 (88.5)	1.426	171.6	0.135	0.269	0.404	0.808	0.192	0.322	0.183
Crushed Granite	3820 (83.0)	1.336	150.2	0.136	0.273	0.409	0.818	0.182	0.308	0.184
Gravel	3970 (86.3)	1.449	147.3	0.134	0.269	0.403	0.806	0.194	0.325	0.182
Average	4034	1.401	152.7	0.135	0.269	0.406	0.811	0.189	0.318	0.183
Per Cent Mortar	(87.7)	(94.2)	(111.8)	(60.5)	(60.3)	.....	(121.2)	(56.9)	(95.8)	(60.8)
1:2:3 CONCRETE (Size 1–1¼)										
White	3580 (77.8)	1.362	148.8	0.136	0.271	0.408	0.815	0.185	0.313	0.184
Red	3550 (77.2)	1.408	146.0	0.135	0.270	0.405	0.810	0.190	0.319	0.183
Slag	3580 (77.8)	1.398	172.5	0.135	0.271	0.405	0.811	0.189	0.318	0.183
Crushed Granite	4175 (90.7)	1.379	149.2	0.136	0.271	0.406	0.813	0.187	0.315	0.184
Gravel	3630 (78.9)	1.389	148.4	0.135	0.271	0.406	0.812	0.188	0.317	0.183
Average	3699	1.387	153.0	0.135	0.271	0.406	0.812	0.188	0.316	0.183
Per Cent Mortar	(80.4)	(93.3)	(112.0)	(60.5)	(60.8)	.....	(121.4)	(56.6)	(95.2)	(60.8)

TABLE 5—VARIATION IN SIZE OF COARSE AGGREGATE: CONCRETES FROM FIVE AGGREGATES: MORTARS CONSTANT (*Continued*)

Coarse Aggregate, Kind	*S*	*V/C*	Unit Wt.	*c*	*a*	*b*	*d*	*V*	*Vm*	*W*
1	2	3	4	5	6	7	8	9	10	11
1:2:3 CONCRETE (Size 1¼–1½)										
White	3790 (82.4)	1.389	148.3	0.135	0.271	0.406	0.812	0.188	0.317	0.183
Red	4050 (88.0)	1.367	146.7	0.136	0.271	0.408	0.815	0.185	0.313	0.184
Slag	3620 (78.7)	1.367	173.0	0.136	0.272	0.407	0.815	0.185	0.312	0.184
Crushed Granite	3860 (83.9)	1.334	150.3	0.136	0.273	0.409	0.818	0.182	0.308	0.182
Gravel	3180 (69.1)	1.416	147.9	0.135	0.269	0.405	0.809	0.191	0.321	0.183
Average	3700	1.375	(153.2	0.136	0.271	0.407	0.814	0.186	0.314	0.184
Per Cent Mortar	(80.4)	(92.5)	112.2)	(61.0)	(60.8)	.....	(121.7)	(56.0)	(94.6)	(61.1)

TABLE 6—VARIATION IN KIND OF COARSE AGGREGATE: CONCRETES FROM SIX DIFFERENT SIZES: MORTARS CONSTANT

Made: Winter, 1924–25. *Specimens:* 6 x 12 in. $W/C = 0.647$ for all. *Age at Test:* 28 days. *Proportions:* Mortar, 1:2:0; Concrete, 1:2:3 by absolute volume. *Fine Aggregate:* Boulder Creek Sand (0–4). *Coarse Aggregate:* Varies (grading and kind). For further data see Tables 1, 2, and 5, and Fig. 5. Same material as Table 5, but differently arranged. See also Notation, Definitions, and Analytical Relations.

Coarse Aggregate, Size	*S*	*V/C*	Unit Wt.	*c*	*a*	*b*	*d*	*V*	*Vm*	*w*
1	2	3	4	5	6	7	8	9	10	11
1:2:0 MORTAR										
None	4480 (97.4)	1.503	136.1	0.222	0.444	.....	0.666	0.334	0.334	0.301
None	4390 (95.5)	1.464	137.3	0.224	0.448	.....	0.672	0.328	0.328	0.304
None	4930 (107.2)	1.493	136.4	0.223	0.445	.....	0.668	0.333	0.333	0.302
Average	4600	1.487	136.6	0.223	0.446	.....	0.669	0.332	0.332	0.302
Per Cent Mortar	(100)	(100)	(100)	(100)	(100)	.....	(100)	(100)	(100)	(100)
1:2:3 CONCRETE (White Sandstone)										
¼–⅜	4710 (102.4)	1.483	146.1	0.133	0.267	0.400	0.800	0.200	0.333	0.180
⅜–½	4140 (90.0)	1.423	147.4	0.135	0.269	0.404	0.808	0.192	0.322	0.183
½–¾	4010 (87.2)	1.422	147.7	0.135	0.270	0.404	0.809	0.191	0.321	0.183
¾–1	4140 (90.0)	1.444	147.0	0.134	0.269	0.403	0.806	0.194	0.325	0.182
1–1¼	3580 (77.8)	1.362	148.8	0.136	0.271	0.408	0.815	0.185	0.313	0.184
1¼–1½	3790 (82.4)	1.389	148.3	0.135	0.271	0.406	0.812	0.188	0.317	0.183
Average	4062	1.420	147.4	0.135	0.269	0.404	0.808	0.192	0.322	0.183
Per Cent Mortar	(88.3)	(95.5)	(107.9)	(60.5)	(60.3)	.....	(120.8)	(57.8)	(97.0)	(60.6)

TABLE 6—VARIATION IN KIND OF COARSE AGGREGATE: CONCRETES FROM SIX DIFFERENT SIZES: MORTARS CONSTANT (*Continued*)

Coarse Aggregate, Size	*S*	*V/C*	Unit Wt.	*c*	*a*	*b*	*d*	*V*	*Vm*	*w*
1	2	3	4	5	6	7	8	9	10	11
1:2:3 CONCRETE (Red Sandstone)										
¼-⅜	4630 (100.7)	1.442	145.2	0.134	0.269	0.403	0.806	0.194	0.325	0.182
⅜-½	4050 (88.0)	1.408	145.8	0.135	0.270	0.405	0.810	0.190	0.319	0.183
½-¾	4000 (86.9)	1.389	146.3	0.135	0.271	0.406	0.812	0.188	0.317	0.183
¾-1	4170 (90.6)	1.348	147.2	0.136	0.272	0.408	0.816	0.184	0.311	0.184
1-1¼	3550 (77.2)	1.408	146.0	0.135	0.270	0.405	0.810	0.190	0.319	0.183
1¼-1½	4050 (88.0)	1.367	146.7	0.136	0.271	0.408	0.815	0.185	0.313	0.184
Average	4075	1.394	146.2	0.135	0.271	0.406	0.811	0.188	0.317	0.186
Per Cent Mortar	(88.6)	(93.8)	(107.0)	(60.5)	(60.8)	.....	(121.2)	(56.6)	(95.5)	(61.6)
1:2:3 CONCRETE (Slag)										
¼-⅜	4970 (108.0)	1.358	173.3	0.136	0.272	0.408	0.816	0.184	0.311	0.184
⅜-½	3990 (86.7)	1.362	173.3	0.136	0.271	0.408	0.815	0.185	0.313	0.184
½-¾	4120 (89.6)	1.390	172.6	0.135	0.271	0.406	0.812	0.188	0.317	0.183
¾-1	4070 (88.5)	1.426	171.6	0.135	0.269	0.404	0.808	0.192	0.322	0.183
1-1¼	3580 (77.8)	1.398	172.5	0.135	0.271	0.405	0.811	0.189	0.318	0.183
1¼-1½	3620 (78.7)	1.367	173.0	0.136	0.272	0.407	0.815	0.185	0.312	0.184
Average	4058	1.383	172.7	0.136	0.271	0.406	0.813	0.187	0.316	0.183
Per Cent Mortar	(88.2)	(93.0)	(126.4)	(61.0)	(60.8)	.....	(121.5)	(56.3)	(95.2)	(60.6)
1:2:3 CONCRETE (Crushed Granite)										
¼-⅜	4400 (95.7)	1.448	148.1	0.134	0.269	0.403	0.806	0.194	0.325	0.182
⅜-½	4270 (92.8)	1.371	149.5	0.136	0.271	0.407	0.814	0.186	0.314	0.184
½-¾	4350 (94.6)	1.353	150.0	0.135	0.273	0.409	0.817	0.183	0.310	0.183
¾-1	3870 (83.0)	1.336	150.2	0.136	0.273	0.409	0.818	0.182	0.308	0.184
1-1¼	4175 (90.8)	1.379	149.2	0.136	0.271	0.406	0.813	0.187	0.315	0.184
1¼-1½	3860 (83.9)	1.334	150.3	0.136	0.273	0.409	0.818	0.182	0.308	0.184
Average	4166	1.370	149.6	0.136	0.272	0.407	0.814	0.186	0.313	0.183
Per Cent Mortar	(90.5)	(92.2)	(109.5)	(61.0)	(61.0)	.....	(121.6)	(56.0)	(94.3)	(60.6)
1:2:3 CONCRETE (Gravel)										
¼-⅜	4470 (97.2)	1.416	147.9	0.135	0.270	0.404	0.809	0.191	0.321	0.183
⅜-½	3890 (83.9)	1.336	149.4	0.136	0.273	0.409	0.818	0.182	0.308	0.184
½-¾	4520 (98.3)	1.436	147.5	0.134	0.270	0.403	0.807	0.193	0.323	0.182
¾-1	3970 (86.3)	1.449	147.3	0.134	0.269	0.403	0.806	0.194	0.325	0.182
1-1¼	4175 (90.7)	1.389	148.4	0.135	0.271	0.406	0.812	0.188	0.317	0.183
1¼-1½	3180 (69.1)	1.416	147.9	0.135	0.269	0.405	0.809	0.191	0.321	0.183
Average	4034	1.407	148.1	0.135	0.270	0.405	0.810	0.190	0.319	0.183
Per Cent Mortar	(87.7)	(94.6)	(108.4)	(60.5)	(60.5)	.....	(121.0)	(57.2)	(96.1)	(60.6)

TABLE 7—MORTAR VS. CONCRETES: MISCELLANEOUS GROUPS: MORTAR CONSTANT

Made: Spring, 1925. *Specimens:* 6 x 12 in. $W/C = 0.647$ for all. *Age at Test:* 28 days. *Proportions:* Mortar, 1:2:0; Concrete, 1:2:3 by absolute volume. *Fine Aggregate:* Boulder Creek Sand (0–4). *Coarse Aggregate:* White Sandstone (quartzite); size varies. For further data see Tables 1 and 2. See also Notation, Definitions, and Analytical Relations.

Spec. No.	S	V/C	Unit Wt.	e	a	b	d	V	Vm	w
1	2	3	4	5	6	7	8	9	10	11
1:2:0 MORTAR (Controls for all series, Spring, 1925)										
6M	5340 (106.0)	1.509	135.9	0.222	0.443	.....	0.665	0.335	0.335	0.301
26M	5630 (122.3)	1.484	136.8	0.223	0.446	.....	0.669	0.331	0.331	0.302
46M	5010 (108.9)	1.508	136.0	0.222	0.443	.....	0.665	0.335	0.335	0.301
16M	5360 (116.5)	1.484	136.8	0.223	0.446	.....	0.669	0.331	0.331	0.302
36M	5220 (113.5)	1.424	138.6	0.226	0.452	.....	0.678	0.322	0.322	0.306
56M	5140 (111.8)	1.484	136.8	0.223	0.446	.....	0.669	0.331	0.331	0.302
66M	5710 (124.1)	1.449	137.8	0.225	0.449	.....	0.674	0.326	0.326	0.305
86M	5070 (110.2)	1.525	135.5	0.221	0.442	.....	0.663	0.337	0.337	0.300
106M	5790 (125.8)	1.488	136.5	0.223	0.445	.....	0.668	0.332	0.332	0.302
76M	5920 (126.5)	1.455	137.8	0.224	0.450	.....	0.674	0.326	0.326	0.304
86M	5130 (111.5)	1.484	136.7	0.223	0.446	.....	0.669	0.331	0.331	0.302
116M	5670 (123.2)	1.488	136.4	0.223	0.445	.....	0.668	0.332	0.332	0.302
Average	5416	1.482	136.8	0.223	0.446	.....	0.669	0.331	0.331	0.302
Per Cent Mortar	(100)	(100)	(100)	(100)	(100)	.....	(100)	(100)	(100)	(100)
1:2:3 CONCRETE (¾–1) (Controls for all series, Spring, 1925)										
4C	4600 (85.1)	1.448	147.0	0.134	0.269	0.403	0.806	0.194	0.325	0.182
24C	4330 (80.1)	1.470	146.6	0.134	0.268	0.401	0.803	0.197	0.329	0.182
44C	4900 (90.7)	1.448	147.0	0.134	0.269	0.403	0.806	0.194	0.325	0.182
14C	4380 (81.1)	1.407	147.8	0.135	0.270	0.405	0.810	0.190	0.320	0.183
34C	4510 (83.4)	1.407	147.7	0.135	0.270	0.405	0.810	0.190	0.320	0.183
54C	4890 (90.5)	1.415	147.5	0.135	0.270	0.404	0.809	0.191	0.320	0.183
64C	4820 (89.2)	1.455	146.8	0.134	0.268	0.403	0.805	0.195	0.327	0.182
84C	4630 (85.7)	1.455	146.9	0.134	0.268	0.403	0.805	0.195	0.327	0.182
104C	5135 (95.0)	1.448	146.9	0.134	0.269	0.403	0.806	0.194	0.325	0.182
74C	5360 (99.2)	1.393	148.2	0.135	0.271	0.406	0.812	0.188	0.317	0.183
94C	5070 (83.8)	1.422	147.4	0.135	0.269	0.404	0.808	0.192	0.323	0.183
114C	5535 (102.2)	1.455	146.9	0.134	0.268	0.403	0.805	0.195	0.327	0.182
Average	4847	1.435	147.2	0.134	0.269	0.404	0.807	0.193	0.324	0.182
Per Cent Mortar	(89.5)	(96.8)	(107.7)	(60.1)	(60.3)	.....	(120.6)	(58.3)	(97.9)	(60.3)
1:2:3 CONCRETE (1–1¼) Batch vs. Single (Average of 23 Specimens)										
Average	4573	1.440	147.1	0.134	0.269	0.404	0.806	0.193	0.325	0.183
Per Cent Mortar	(84.5)	(97.2)	(107.5)	(60.1)	(60.3)	.....	(120.5)	(58.3)	(98.2)	(60.6)

TABLE 8—MORTARS VS. CONCRETES FOR VARIABLE CURING CONDITIONS: CONSTANT MORTAR

Made; Spring, 1926. *Specimens:* 6 x 12 in. $W/C = 0.90$ for all. *Proportions:* Mortar, 1:2:0; Concrete, 1:2:3 by actual loose volume. Proportioning was done by careful weighing, but an "actual loose volume" determination was made to determine the weights to be used. *Fine Aggregate:* Boulder Creek Sand (0–4). *Coarse Aggregate:* Red Sandstone (¾–1). For further data see Tables 1 and 2. The data for standard curing only are plotted on Fig. 8.

NOTE.—The age is the age at test for most specimens. For those moist cured but dried out before test, the age is the period of moist curing. In this laboratory 6 x 12 in. cylinders dry in air to constant strength in about 12 or 15 days. After that several months in air has little or no added effect. (See *Proceedings*, A. C. I., 1926, pp. 421 and 424.)

Curing	Age	Strength, lb. per sq. in.		Per Cent Strength		No. Tests Averaged	
		Concrete	Mortar	Concrete of Mortar	Mortar of Concrete	Concrete	Mortar
1	2	3	4	5	6	7	8
Standard (in water, wet at test)	1 day	290	....	....	.....	2	..
	4 days	1190	1307	91.0	109.9	2	4
	7 days	1635	1933	84.6	118.2	2	2
	28 days	2935	3604	82.0	122.0	10	9
	4 mos.	4089	4710	86.9	115.2	3	3
	8 mos.	4075	5055	80.7	124.1	2	2
Average				85.0	117.9		
Wet Cured. Dry at test	4 days	3100	....	....	.....	1	..
	12 days	3533	....	....	.....	3	..
	25 days	4220	....	....	.....	1	..
	4 mos.	5275	6430	82.1	121.9	2	2
	7 mos.	5207	6500	80.2	124.8	3	2
Average				81.2	123.3		
Air Cured. Dry at test	4 days	1177	....	....	.....	1	..
	7 days	1520	....	....	.....	1	..
	28 days	2440	2770	88.1	113.5	3	1
	4 mos.	2451	2870	85.5	117.1	2	1
	8 mos.	2310	....	....	.....	2	..
Average				86.8	115.3		
Air Cured. Wet at test	4 days	1132	....	....	.....	1	..
	7 days	1288	....	....	.....	1	..
	28 days	1869	....	....	.....	6	..
	4 mos.	1910	2200	86.9	115.2	1	1
	8 mos.	1820	....	....	.....	1	..
Average				86.9	115.2		
Wet Cured. Oven Dried. Dry at test	4 days	2538	....	....	.....	3	..
	12 days	2920	....	....	.....	5	..
	25 days	3075	4400	69.9	143.2	7	2
	4 mos.	4190	....	....	.....	1	..
Average				69.9	143.2		
Oven Dried at 1 day. Air till test. Tested Dry	4 days	1530	1795	85.3	117.3	5	2
	7 days	1601	....	....	.....	4	..
	28 days	1761	....	....	.....	10	..
	4 mos.	1808	....	....	.....	1	..
	8 mos.	1920	....	....	.....	1	..
Average				85.3	117.3		
Grand Average for all conditions				83.6	120.2		

Discussion—Aggregate as Field for Needed Research.

Mr. Wheeler.

Walter H. Wheeler (*By Letter*).—I agree with Mr. Gilkey thoroughly that there is room for much needed investigation along this line, both as to the effect of the coarse aggregate upon the strength of the concrete and the effect of the coarse aggregate upon the durability of concrete under wear and abrasion and as to its durability under weathering and frost action.

I have seen a number of examples of otherwise fine concrete structures in northern climates and also in temperate climates where the temperature rarely goes near the zero mark, which after a few years had seriously disintegrated even to the point of endangering these structures. I am convinced by my observations and investigations that in at least a number of these instances these failures are directly chargeable to the quality of the coarse aggregate. It is my experience that durable concrete cannot be made from coarse aggregate which is not durable in itself. There are coarse aggregates which meet all the requirements for good concrete except the requirement that they offer satisfactory resistance to the action of frost. It is my observation that concrete made from such aggregates will test up very well for strength and durability until the frost gets in its work and then it quickly disintegrates. On all work to be exposed to the weather, I have made it a practice to submit the coarse aggregates to the freezing or sodium sulphate test whenever there was any question as to their ability to resist frost, and other means of determining the question were not practical such as examination of the weathered rock ledges.

A total of 76,000 cu. yd. of concrete were used in the construction of the Mendota Bridge. In addition to accurate sizing and grading, my specification required that the coarse aggregate have a hardness and toughness of 8 and water absorption of less than 1 lb. per cu. ft. This requirement eliminated the aggregates which were available which would not meet the frost test. Gravel aggregate was used for all the concrete in the super-structure. In the foundations some of the concrete was made with crushed rock aggregate. The maximum size of aggregate for pier concrete was 1½ in. and for superstructure was 1 in. In the fall of 1925 the inundation system was installed. About three-fifths of the concrete had been poured before that time. The total amount of water used in the concrete prior to installing inundation ranged from 6½ to 6¾ gal. per sack of cement. The test cylinders of this concrete at 28 days averaged 2,707 lb. per sq. in. and ranged from 2,413 to 2,824 lb. per sq. in. Only

one cylinder tested below 2,600 lb. per sq. in. The average 7-day test for the crushed rock concrete was 1,840 lb. per sq. in. The average 28-day test for crushed rock concrete was 2,709 lb. per sq. in. There were only a few cylinders of the crushed rock concrete but these tests indicate a higher 7-day strength for the crushed rock and no higher 28-day strength.

After inundation was installed it was possible to reduce the water to 6¼ gal. per sack of cement and the 28-day tests of gravel concrete averaged 3,127 lb. per sq. in. and ranged from 2,672 lb. per sq. in. to 3,521 lb. per sq. in. The effect of inundation, besides reducing the water required, was to make a better mixture in the same length of time. The concrete was mixed a minimum of 1½ min. but rarely less than 2 min. actual time. Sometimes it remained in the mixer 4 or 5 min. I found that with inundation the concrete seemed to be as well mixed in 2 min. as it was in 3 to 4 min. without inundation.

In paving the approach roads gravel aggregate was used—maximum size, 2½ in. About 4½ gal. of water were used per sack of cement. About 80,000 sq. yd. of concrete pavements were laid. The test cylinders taken on this work averaged 2,370 lb. per sq. in. at 28 days and ranged from 1,766 lb. to 3,263 lb. per sq. in. The inundation system was not used on this work but the water was accurately measured, and the aggregates were accurately measured in batchers. The minimum time of mixing was 1½ min. The paving was done in July, August, September, October and November, 1925, and cured 7 days with wet earth covering. In July, 1926, six cores were drilled from this pavement at intervals of about 1 mile. These cores were broken in compression and averaged 4,998 lb. per sq. in. and ranged from 4,070 to 6,460 lb. per sq. in. The corrected average unit stress was 4,771 lb. per sq. in.

Mr. Gonnerman

H. F. GONNERMAN* (*By Letter*).—Prof. Gilkey has raised a number of important questions concerning the function of the coarse aggregate in concrete and presents some experimental data from which the conclusion is drawn that in mortars and concretes of the same void-cement ratio or water-cement ratio, the compressive strength of the mortar is greater than that of the concrete. The author has correctly stated the water-cement ratio law of Abrams in his introductory remarks but in his tests has ignored the necessary requirement that the mixes must be both plastic and workable. The water-cement ratio law is, that for plastic and workable mixes the strength and other qualities of concrete are fixed by the proportion of water to cement. Under this law, to be sure, a mortar of a given water-cement ratio would have the same strength as a concrete of the same water-cement ratio provided both were within the proper range of plasticity and workability. This does not mean, for example, that a 1:2 mortar of any water-ratio will have the same strength as a 1:2:4 concrete of the same water-ratio, because both mixes will not be workable for the same water-ratio; either the mortar will be so wet as to permit segregation or the concrete will be too stiff to puddle and therefore will not be plastic.

* Manager, Research Laboratory, Portland Cement Association.

A critical study of the data presented by Prof. Gilkey reveals in many instances the non-plasticity of the mixtures in his tests which, in the opinion of the writer, accounts for the results he obtained. For example, all of the mixtures in Table III leaner than 1:2:2 were unworkable due to the small amount of mixing water. Since the publication of the paper the author has supplied the writer with the slumps shown by many of the mixes. In this table none of the mixes leaner than the 1:2:2 gave slumps exceeding 1.1 in. and six of them gave slumps less than ½ in. The author states that these mixes were workable but stiff. The experience of the research laboratory of the Portland Cement Association has been that for mixes of this consistency the strengths are invariably low.

Another feature that will account for some of the low strengths with the larger sizes and proportions of coarse aggregate is the size of the test piece. For example, in certain of the mixtures in Table IV coarse aggregates ranging from 1½ to 3 in. in size were used in a 1:2:4 mix (by absolute volume) and the low strengths obtained with this size of aggregate may be attributed to the use of 6 x 12-in. cylinders as test pieces. The 6 x 12-in. cylinder is not satisfactory for testing aggregates larger than 2 in. in diameter.

The slumps reported for the concrete specimens in Tables V and VI show that in practically every case the concrete was so dry as to be unworkable. Furthermore, the data in these two tables, except for the mortars, were based on tests of a single specimen for a given condition, and are therefore insufficient as to serve as a basis for such an important conclusion as Prof. Gilkey has drawn. Allowance must be made for experimental errors as it is not uncommon to have extreme variations in strength of laboratory specimens made under the same conditions of as much as 15 to 20 per cent of the average strength. The dry mixes are much more erratic in this respect than the more plastic mixes. It is interesting to note that when the concrete strengths for the different sizes and kinds of coarse aggregates in Tables V and VI are compared with the *lowest* of the three mortar strengths (4,390 lb. per sq. in.) the agreement between mortar and concrete strengths is well within the limit of experimental error.

Another element entering into Prof. Gilkey's results which is equally as important as those already brought out in explaining the higher mortar strength, is the loss of water during the filling of the mold and in the short period thereafter while the mixture is still plastic. In the very wet mixes which he used in many of his mortars, this loss may be considerable while in the stiff mixes like those used in the concrete specimens, it is nil. The loss of ½ pint of water from a 6 x 12-in. cylinder of the mortars used in these tests would result in a strength about 600 lb. per sq. in. higher than that corresponding to the nominal water-cement ratios used. This is about equal to the greatest difference shown in the author's data.

The author suggests as a needed slogan in reference to the making of concrete tests "Not how many but how well." In this connection it may be pointed out that an understanding of some of the fine points in making

tests such as have been referred to is one of the byproducts of experience with large numbers of tests and in testing, as in other lines, it is the proper use of byproducts that often spells the difference between success and failure.

Our laboratory has made a great many tests on mortars and concretes which have a bearing on the question raised by Prof. Gilkey and some data from a recent series may be of interest. In one group of tests, two sands from Elgin, Ill. (0-No. 14 and 0-No. 4) were used singly and in combination with coarse aggregate consisting of Elgin pebbles or Chicago crushed limestone graded No. 4-1½ in., or ¾-1½ in. Each of the aggregate combinations was used in 1: 7, 1: 5 and 1: 3 mixes, sufficient water being used with each mix to give flows of 150, 190 or 220 as measured by the flow table; slump tests were also made using the standard 4 x 8 x 12-in. cone and the workability or non-workability of the mixtures was judged by an experienced operator. Five specimens were made for each condition on five different days following A. S. T. M. standard procedure. A total of 306 mixtures or 1,530 specimens were tested. Following is a skeleton outline of the tests:

Aggregate			Fineness Modulus of Mixed Aggregate	Mix by Volume	Flow
Fine	Coarse				
	Size	Kind			
0–No. 14 Elgin Sand	4–1½ in.	Elgin Pebbles	2.20 3.10 4.65 5.65	1:7 1:5 1:3	150 190 220
0–No. 4 Elgin Sand	¾–1½ in.	Chicago Limestone			

The data for the crushed limestone coarse aggregate in this group of tests are plotted in Fig. 1, which shows the relation between compressive strength and water content. All of the tests in the group are included in this diagram, except those which had a slump of 1 in. or less or which were too harsh to puddle to a smooth finish. Several of the strength values for the different sizes of aggregate were coincident and in such cases strengths for one size only are shown, so that there were somewhat more tests than indicated by the number of points on the diagram. Absorption of water by the aggregate was determined and the water contents of the mixes corrected accordingly in plotting Fig. 1. The data for the pebble coarse aggregate gave a curve identical with that in Fig. 1 and because of this similarity is not included.

It will be seen from Fig. 1 that the strengths of the mortars and concretes lie close to the curve drawn through all the points, showing that the strength of mortar is the same as that of concrete for similar water contents. Furthermore, for the same water-cement ratio, the strengths of concretes from coarse aggregate graded ¾ to 1½ in. in size are the same as those from 4 to 1½-in. aggregate—a result contrary to that obtained by Prof. Gilkey.

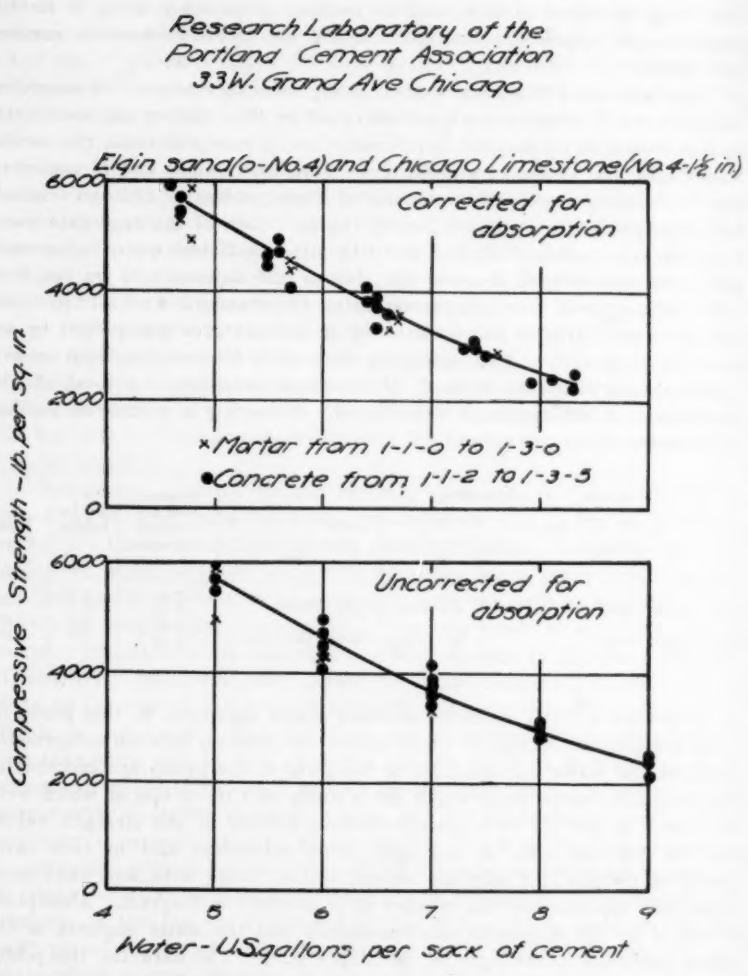

FIG. 1.—RELATION OF COMPRESSIVE STRENGTH OF MORTAR AND CONCRETE TO QUANTITY OF MIXING WATER.

Compression tests of 6 x 12-in. cylinders.
Cylinders cured in moist room until test; tested damp.
Age at test, 28 days.
Each value is the average of five tests made on different days.

In another group of tests, five hundred and thirty 6 x 12-in. mortar and concrete cylinders were made using 5, 6, 7, 8, or 9 gal. of water per sack of cement and five ratios of cement to sand by volume as follows:

Ratio of Cement to Sand	Mix by Volume				
	U. S. Gallons of Water per Sack of Cement				
	5	6	7	8	9
1:1	1:1:0	.....	.....	.....	.....
	1:1:2	1:1:2	.....	.....	.....
	1:1:3	1:1:3	.....	.....	.....
1:1½	1:1½:0	1:1½:0	.....	.....	.....
	1:1½:2	1:1½:2	1:1½:2	.....	.....
	.....	1:1½:3	1:1½:3	1:1½:3	.....
	.....	1:1½:4	1:1½:4	1:1½:4	.....
1:2	1:2:0	1:2:0	1:2:0	.....	.....
	1:2:2	1:2:2	1:2:2	.....	.....
	.....	1:2:3	1:2:3	1:2:3	.....
	.....	.....	1:2:4	1:2:4	1:2:4
	.....	.....	1:2:5	1:2:5	1:2:5
1:2½	1:2½:0	1:2½:0	1:2½:0	.....	.....
	.....	1:2½:3	1:2½:3	1:2½:3	.....
	.....	.....	1:2½:4	1:2½:4	1:2½:4
	.....	.....	.....	1:2½:5	1:2½:5
	.....	.....	.....	.....	1:2½:6
1:3	1:3:0	1:3:0	1:3:0	.....	.....
	.....	1:3:2	1:3:2	.....	.....
	.....	.....	1:3:3	1:3:3	.....
	.....	.....	1:3:4	1:3:4	.....
	.....	.....	.....	1:3:5	.....

The fine aggregate was Elgin sand graded 0-No. 4 and the coarse aggregate Elgin pebbles or Chicago limestone graded No. 4-1½ in. The consistency of the mixtures was measured by means of the flow table and slump cone and the workability or non-workability judged by an experienced operator.

The compressive strengths of the *workable mixes* (proper and ratios and slumps 1 in. or over) from limestone coarse aggregate are plotted in Fig. 2 for the various water contents both uncorrected and corrected for absorption by the aggregate; similar curves were obtained for the pebble coarse aggregate and are not included. These tests, like those in Fig. 1, show that the strength of mortar or concrete is controlled by the water content of the mix and that for similar water contents the strength of mortar is the same as that of concrete, provided that the mixtures are plastic and workable.

Many other tests at this laboratory can be cited which show the same close agreement with the water-cement ratio strength law. While in the very earliest publications of Prof. Abrams the importance of workability was emphasized, the significance of this factor was not fully realized until more recently. Some of the discrepancies in the earlier tests are now known to have been due to a departure from the field of truly workable mixes. And by this term is meant those mixes that are neither too harsh

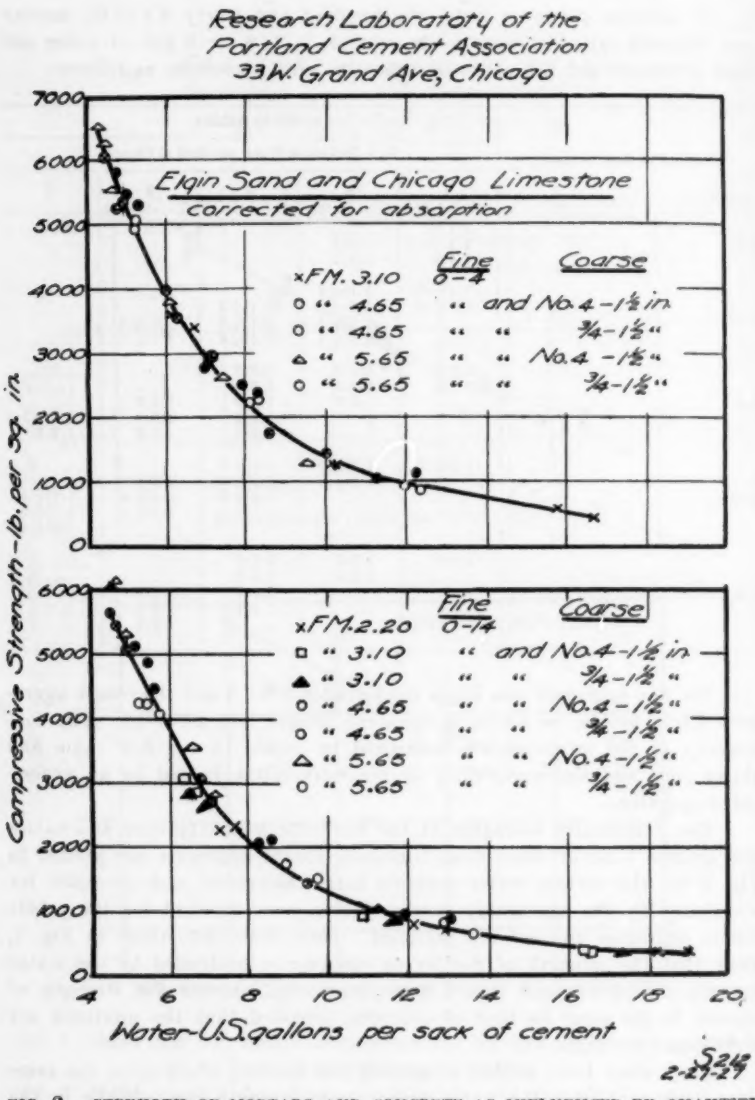
Research Laboratory of the
Portland Cement Association
33 W. Grand Ave, Chicago
Elgin Sand and Chicago Limestone
corrected for absorption
Fine 0-4
Coarse
×FM. 3.10 " and No. 4-1½ in.
○ " 4.65 " " ¾-1½ "
○ " 4.65 " " No. 4-1½ "
△ " 5.65 " " ¾-1½ "
○ " 5.65 " "
Fine 0-14
Coarse
×FM. 2.20 " and No. 4-1½ in.
□ " 3.10 " " ¾-1½ "
▲ " 3.10 " " No. 4-1½ "
○ " 4.65 " " ¾-1½ "
○ " 4.65 " " No. 4-1½ "
△ " 5.65 " " ¾-1½ "
○ " 5.65 " "
Compressive Strength-lb. per sq. in.
Water-U.S. gallons per sack of cement
S248
2-27-29

FIG. 2.—STRENGTH OF MORTARS AND CONCRETE AS INFLUENCED BY QUANTITY OF MIXING WATER.

Compression tests of 6 x 12-in. cylinders.
Cylinders cured in moist room until test; tested damp.
Age at test, 28 days.
Each value is the average of five tests made on different days.

nor too stiff to puddle on the one hand, nor too wet as to permit segregation on the other. When measured by this test for workability the results from Prof. Gilkey's tests will be found to give very different conclusions from those he has drawn.

G. M. WILLIAMS (*By Letter*).—The results of tests presented by Professor Gilkey add to the evidence which indicates that the *w/c* relation alone does not account for differences in compressive strength of concrete. While it is admitted that the water-cement relation is the greatest single factor, the grading of the aggregate, its maximum size and its surface condition cannot be ignored if best use is to be made of available aggregates. Mr. Williams.

The addition of fine aggregate to neat cement pastes having equal water-cement ratios produces mortars having lower compressive strengths in the same manner that coarse aggregates added to identical mortars reduce strengths as illustrated in this paper. The effect is similar and the results are consistent whether the added aggregate be smaller or larger than that size which has been arbitrarily established as the dividing line between fine aggregate and coarse aggregate. It has been the writer's experience that limiting the maximum size of coarse aggregate to that passing the ¾-in. screen will generally furnish concretes equal to if not superior to concretes containing aggregate up to the usually specified 1½-in. size. In localities where the natural deposits lack this larger size the expense of bringing in additional coarse material can well be avoided without reduction in concrete quality.

It has been the practice in some cases of mass concrete construction, such as dams, foundations and footings to add large aggregate of "plum" stones to the concrete already in place. Such procedure will usually reduce the cost of concrete per cubic yard and it is generally assumed that a concrete of higher strength will result. The assumption of higher strength may be based upon absorption of water from the concrete by the added stone, upon the increase in density or solidity ratio which results, or upon the addition of material of much higher compressive strength to the weaker concrete. Laboratory test results consistently show that any increase which might be expected due to absorption of water is more than counterbalanced by the reduction in strength due to the presence of the large aggregate masses. The net result is a concrete of lower strength. The density increase may be of value in that a heavier concrete is obtained and this factor alone may justify the use of "plum" stones in dams or large masses where compressive strength is to a certain degree secondary and greater weight is of first importance.

On some work, where the maximum size aggregate put through the mixer ranged up to 6 in. or more in size, a dimension which precludes the molding of specimens small enough to be crushed in available testing machines, tests have been made upon specimens from which particles larger than 1½ in. have been removed. This test concrete is not representative of that placed in the structure and the strengths so obtained will be appreciably higher than the true strengths.

With reference to the relation between the compressive strength of concrete and the compressive strength of aggregate the writer has in mind tests in which crushed clay bricks having a compressive strength of about 1,000 lb. per sq. in. were used as coarse aggregate in comparison with good quality gravel. The brick aggregate furnished concrete having a strength of about 3,000 lb. per sq. in., as did the gravel aggregate for the same cement content. It cannot be argued from such results that strength of aggregate is of little or no importance, but such tests emphasize the author's argument that much is yet to be learned as to the functions and properties of aggregates, as opposed to the generally accepted idea that strong aggregates are required to make strong concrete. That surface condition of coarse aggregate may alone be a factor affecting strength of concrete was pointed out in a series of tests in which the coarse aggregate was divided into two lots and after treatment incorporated into identical mortars. One lot of coarse aggregate was saturated to prevent absorption of moisture from the mortar and surface dried just before use. The second lot was warmed and placed in molten paraffine and transferred immediately after to a coarse screen and shaken to remove as much paraffine as possible. After cooling the resultant paraffine film was so slight as to be apparent only under close inspection. The paraffined aggregate furnished concretes approximately 70 per cent as strong as the normal aggregate.

Mr. Talbot. A. N. TALBOT.—The paper of Prof. Gilkey brings out the fact that there are many variables that affect the properties of concrete, not only strength, but others, and that we should not overlook one in our desire to correct evils from another. We have had our attention called, quite properly, to the abuse in the use of water and the importance of keeping down the ratio between cement and water. This is only one factor, though an important one. We are probably not paying enough attention to the quality of the aggregate, particularly to the fine aggregate and also we are careless in the matter of the coarse aggregate. Sufficient attention is not being given to matters connected with the placing and the effect of the placing upon strength, permanence and other qualities of concrete. I have said with reference to the relation between mortar strength and the strength of concrete, that, other things being equal, the strength of the mortar controls the strength of the concrete. But other things are not always equal and I think one of these is that brought up by Prof. Gilkey, the gradation of the coarse aggregates and the effect of size of the aggregate. Those are matters which we should know more about. It is the common idea abroad that the larger the coarse aggregate the stronger the concrete. Tests show that that is not true except under very limited conditions. It has been held that the gradation of the coarse aggregate does not matter provided the amount used is the same and provided the water-cement ratio was kept the same. Tests made in different laboratories, including later tests at the University of Illinois, indicate that that is not true, particularly if workability be taken into account. I am glad, then, to see this paper brought out. It suggests the need of considering all the aspects of the problem of making concrete.

HERBERT J. GILKEY (*By Letter*).—The writer appreciates the number, quality, and character of the discussions offered. The nature of the response encourages him to feel that the subject is a timely one in the minds of others as well as himself. Mr. Gilkey.

The scope of this closure will be largely restricted to an attempt to straighten out some of the points raised by Mr. Gonnerman in his fine tooth comb attack. The good faith and sincerity of Mr. Gonnerman are unquestioned but he does enter the fray as if something sacred had been profaned.

It is granted that the comparison of concretes with mortars of the same sand and water ratios is not entirely satisfactory and that the two water-cement ratio vs. strength curves can have but a partial overlap. The wetter portion of the mortar can overlap the drier workable range of the concrete as is shown in Fig. 7. The respective strengths for any consistency can always be compared, however, with that given by the Abrams law $S = \frac{14{,}000}{7^x}$.

If the water ratio be kept constant but the sand ratio increased to attain comparable stiffness, the mortar strength will soon fall below that of the concrete instead of lying above it. Professor Williams in his discussion cites tests in which aggregate in varying amounts was added to a neat cement paste of constant water ratio. Several months ago the writer ran such a series, using standard sand, with 2 x 4-in. specimens. The findings taken from the plotted test results are as follows:

Variation in Amount of Standard Sand
(28-day strengths, standard curing, 2 by 4 in. specimens)

Mixture	Water-Cement Ratio					
	0.30	0.40	0.50	0.60	0.70	0.80
Neat	9400	8700	7800	6500	5500	4600
1:1	too dry	8500	7800	6500	5500	4600
1:1½	"	7000	6500	5600	4700	3800
1:2½	"	4800	4700	4300	3900	3200
1:3	"	too dry	too dry	3300	3300	2900
1:3½	"	"	"	too dry	2600	2400
1:4½	"	"	"	"	2000	1900
1:5	"	"	"	"	too dry	1300
Abrams' Upper Curve	....	6500	5300	4400	3600	3000

It is interesting to note, in passing, that the 1:1 mixture of these tests gave the same strengths as the neat cements and that the 1:2½ agrees with Abrams' upper strength curve for water ratio of 0.60 and upwards. Whether these are general or merely incidental phenomenon cannot be said.

In the body of the paper the writer went to great length to show that good specimens were obtained in *every instance*. During the past few years the writer has read and heard enough of water ratio controversy to have been fully aware that his findings would be assailed from the angles of freak mixtures, non-plasticity, and lack of workability. He knew that zero slumps would be pounced upon as indications of utter unworkableness,

Pains were taken, therefore, to explain and apply two independent tests for workable mixtures. A low or zero slump is a very proper basis for suspecting non-workability but the strength and density are the final measures of whether or not the concrete was properly placeable.

What is workable concrete for some operators and methods of placement would prove to be a friable weak and honeycombed product in the hands of others. In all dry mixtures (less than 2-in. slump) the writer used an air vibrator against the sides of the mold and was able, in every case, to bring moisture to the surface of each successive layer by capillary action. This is a reasonably infallible sign that adequate placement is being achieved. The vibrator has no effect on the strength or properties of the concrete that differs from that of thorough tamping or pounding of the mold. It greatly facilitates placement of mixtures that would otherwise be difficult or impossible to successfully handle. Another point of utmost importance in the making was the fact that not one scoop of material entered any mold in the writer's absence from the tamping rod or the vibrator. Such close personal supervision would not be possible in a large laboratory *making many thousands of specimens.* The writer can sympathetically understand Mr. Gonnerman's terror of the zero slump mixture for he, too, in times past has endeavored to have his assistants fill the molds. In the small laboratory there is, perforce, a closeness to the job that the wholesaler may well envy.

If Mr. Gonnerman will select those specimens that he so freely brands "unworkable" and compare the voids in the mortar (V_m) with those in the plastic mortar of the same water-cement ratio he will fail to find the increase in voids that denotes poor placement. If he will compare the strengths of these specimens with those from the orthodox Abrams' curve he will detect in this "unworkableness" a good substitute for cement. As stated and re-stated in the paper, the only evidence of non-agreement in strengths of these concretes and mortars is that of extra strength in the mortars and not of weakness in the concretes. It is singular that such minute thoroughness as Mr. Gonnerman has displayed could have overlooked these real tests as to whether or not the concrete was workable as placed. And this on top of emphasis and re-emphasis of the points in question! Truly this savors of "the blindness that will not see."

SUMMARY OF TABLE III
Effect of Variation in Quantity of Coarse Aggregate (Mortar Constant).
Average of 3 specimens each proportion.
$W/C = 0.647$ (4000 lb. per sq. in.)

Column	2	3	4	5
Mixture	Mean Slump	Mortar Voids (V_m)	Strength	Per Cent Abrams
1:2:0	3.1	0.330	5573	140
1:2:½	4.1	0.332	5150	129
1:2:1	3.2	0.334	4970	124
1:2:1½	1.7	0.327	5000	125
1:2:2	1.5	0.327	4743	118
1:2:2½	1.0	0.322	4765	119
1:2:3	0.9	0.323	5047	126
1:2:3½	0.2	0.328	4847	121
1:2:4	0.1	0.323	4690	117
Abrams Strength for $W/C = 0.647$			4000	100

This table speaks for itself. Note the uniformity of the mortar voids, the driest mixture containing slightly fewer voids than the others. Note the strength comparisons and the percentages of the 4,000 lb. from Abrams' upper curve. Is there evidence of weakness or porosity in these low-slump concretes?

ABSTRACT OF TABLE IV

Size and Quantity of Coarse Aggregate

$W/C = 0.90$ (2450 lb. per sq. in.)

2 specimens each.

Column	2	3	4
Size	Proportions		
	1:2:3	1:2:3.5	1:2:4
¼–½	3448	3360	3030
½–¾	3356	3405	3156
¾–1½	3059	2831	2335
1½–3	2694	2280	2067
Mortar (average of 3)	3799		
Abrams for $W/C = 0.90$	2450		

Note that the 1:2:4 with 1 to 1½-in. aggregate corresponds closely to Abrams' upper curve $S = \frac{14{,}000}{7^x}$. All richer mixtures and smaller aggregates fall well above it. This represents a very usual mixture and probably predominated in the tests from which Abrams' laws were formulated.

Note the distinct drop in strength between the ½ to ¾ and the 1 to 1½. Disregard the 1½ to 3 entirely and the trend is unchanged.

The writer agrees that 1½ to 3 is a large aggregate for 6 x 12-in. specimens and does not pretend to offer the findings from this size aggregate as final. Although called 1½ to 3 probably 90 per cent of these stones would have passed a 2-in. mesh. To seriously proffer this post-mortem extenuation is quite unnecesary. For each one of the three proportions the trend seems to be definitely indicated below the 1½-in. screen. The results from the largest size may be seasoned to suit the individual taste or struck from the menu entirely without affecting the nature of the evidence presented in Table IV.

That the elimination of the 1½ to 3 data is probably not warranted would be indicated by the fact that each pair of strengths for this size of aggregate have as close an agreement as the results on any of the other aggregate sizes. This can be noted by reference to the unabridged Table IV in the text of the paper. There is lack of evidence of the erratic showing that Mr. Gonnerman says should be expected when the size exceeds 2 in. in 6-in. specimens.

Tables V and VI (the same data in two different arrangements) are criticised on several counts, viz.:

1. Non-workability (as indicated by low slumps) as usual.

2. Only one concrete specimen per test condition.
3. Concrete strengths compare favorably with the *lowest* of the three mortar strengths.

ABSTRACT OF TABLES V AND VI

Comparison of 6 different one-size gradings of 5 different coarse aggregates.
1 specimen per recorded strength.
Mortar constant. $W/C = 0.647$ for all .
(Strength by Abrams' upper curve should be constant at 4000 lb. per sq. in.)
All concretes 1:2:3 by absolute volume.

Column.....	2	3	4	5	6	7	8	9	10
Mortars.....	4480	4390	4930	....	....	4600	115	0.332	7.0
Concretes.....	White	Red	Slag	Crushed Granite	Gravel	Average	Per cent of Abrams	Mortar Voids (Vm)	Average Slump
¼–⅜.....	4710	4630	4970	4400	4470	4636	116	0.323	0.0
⅜–½.....	4140	4050	3990	4270	3890	4068	102	0.315	0.1
½–¾.....	4010	4000	4120	4350	4520	4200	105	0.318	0.3
¾–1.....	4140	4170	4070	3820	3970	4034	101	0.318	1.1
1–1½.....	3580	3550	3580	4175	3630	3699	92	0.316	1.9
1½–1½.....	3790	4050	3620	3860	3180	3700	92	0.314	1.9
Abrams Strength $W/C = 0.647$						4000	100		

Objection No. 1 is fully and eloquently answered in the usual manner by the data of columns 7, 8 and 9. Remembering that these concretes and mortars are all of constant water ratio is it not illuminating to note the high strengths given by "unworkability"? The concretes that Mr. Gonnerman would exclude on this count (slumps of 0.0 to 1.1, aggregates below 1 in.) all show strengths well above the Abrams upper curve. Those that he accepts fall far below it. Column 6 shows the concretes as having somewhat denser, and therefore stronger mortars, than the mortars with which they are compared.

Passing to objection No. 2, we find, instead of one specimen of a kind, five each in the size comparison and six each in the comparison of kinds of aggregate. Is not the result on the effect of size as expressed by tests on five different aggregates of that size to be considered the full equivalent of, if not superior to, five tests of that size performed on a single kind of aggregate? In like manner is not a comparison of aggregates in six different gradings the full equivalent of six companion specimens of one grading? These results are rather less concordant than most of the test results of this paper, but there is certainly no doubt as to the trend indicated even though the path be a little irregular. Mr. Gonnerman himself comes to the rescue in explaining that any one specimen may be erratic and it would appear that those few in which he may here find solace, belong to that class. Since Mr. Gonnerman further explains that "the dry mixes are more erratic than the more plastic ones," the weakest of the concretes (slumps of 1.9 in., aggregates above 1 in.) should by implication be given the greater weight in the comparisons made. Their story is quite consistent. Relative to Tables V and VI and the alleged single specimen of

a kind Mr. Gonnerman writes "and are therefore insufficient to serve as a basis for such an important conclusion as Prof. Gilkey has drawn." No conclusions have been drawn from Tables V and VI. These data are corroborative of those of other tables and from the whole assembled mass certain tentative findings have been offered, as evidence that a real and comprehensive investigation should be launched. Neither vindication nor substantiation is sought. The writer has no printed utterances to defend, no pet theory to support. He has no dread of retractions. The findings here set forth are of a purely reconnaissance and tentative nature and although there are apparent many strong and definite indications, or trends, nothing is considered as having been proved. Nothing would please the writer more than to see the correctness of the water-cement ratio theory authoritatively demonstrated for *all workable proportions and gradings* of aggregates. No friend of concrete cares to clutter the art with undue refinements or any irrelevant or false complications.

As to the third point launched in connection with Tables V and VI, the writer does not understand why Mr. Gonnerman, exponent of averages, should suddenly look longingly at the lowest of the mortar strengths as a correct basis for comparison. The lowest mortar strength of 4,390 may, in the writer's estimation, be very properly compared with the lowest concrete strength of 3,180. Surely Mr. Gonnerman is not implying that the weakest mortar specimen should be compared with the strongest concrete!

Mr. Gonnerman brings out an interesting and appealing point in mentioning the water loss that occurs from leakage after specimens are made and prior to initial set. Any one who has thoughtfully observed this phenomenon will probably agree to the following as being correct statements of fact:

1. In comparing concretes and their mortars, the mortars are much thinner than the concretes.
2. Leakage does occur *in both* at the contact of mold and base plate.
3. The thin mortar loses more water than the stiffer concrete.
4. The water lost is clear and free from cement and the water-cement ratio is therefore lowered; with corresponding strength gain.
5. Shrinkage occurs as evidenced by settlement below the top of the mold if the specimen was finished flush.
6. In 6 x 12-in. specimens in our laboratory shrinkages have measured as much as ½ in. for very wet mortars. It might be more with some equipment. Our molds give practically no leakage along the vertical joint and we use an oiled paper between mold and machined base plate. For concretes as wet as the mortars (a higher water-cement ratio, or less sand, of course) the shrinkage will be less than that of the mortar. For a concrete of the same water-cement ratio as the mortar, one that seems stiff in comparison with the mortar, the shrinkage may be scarcely noticeable.

Early in his comparisons of mortars and concretes, the writer observed these facts and he reacted to them much as has Mr. Gonnerman. Further thought along these lines has, for the writer at least, placed quite a differ-

ent construction upon them. Suppose we consider a stiff concrete composed half of stone and half of mortar. (Few concretes contain much more than 50 per cent of coarse aggregate.) The mortar of this particular concrete will be very fluid and will give relatively high leakage and considerable shrinkage. With exactly half the water loss of the mortar, the concrete will have undergone the same change in water-cement ratio since it contains just half as much cement as does the mortar cylinder. Is the concrete likely to lose half as much water as the mortar? Why shouldn't it? Will a pile of coarse aggregate retain any appreciable quantity of free water? The water that coarse aggregate will retain is negligible as compared with the water retaining properties of sand and cement. Is there any reason to believe that coarse aggregate embedded in a mortar matrix will retain any more moisture than it would out in a pile by itself? Or is there any mysterious process whereby the moisture-retaining propensities of sand and cement will be increased by the presence of these larger fragments that we call coarse aggregate?

So much for a theoretical consideration of the phenomenon. Rather thorough experimental checking has been conducted on some 350 specimens and there seems to be little question but that the net effect of moisture losses of this sort on effective water-cement ratios are negligible in their effect on comparative strengths. The measurement of losses was accomplished by weighing the mold and specimen a second time, after water loss had ceased and just prior to capping. The difference between this weight and that as finished gave a true measure of water lost by leakage and evaporation. The experimental differences sometimes favored the mortar and sometimes the concrete. Even very dry concretes had measurable losses.

The concretes do not necessarily undergo their proper proportion of shrinkage which fact is easily explained by the supporting rigid framework of coarse aggregate.

There is another by-product of this phenomenon that is important but the effect of which is as yet unevaluated. The segregation and formation of laitance and water gain is probably much more pronounced in the mortar specimen than in the concrete one. The mortar is held by a supporting network of stone in the latter. Thus the only aspect of Mr. Gonnerman's contention that may have a measurable effect upon the strength favors the concrete and not the mortar. Absorption, to whatever extent present, is another factor contributing toward a lower mortar strength.

Mr. Gonnerman's estimate of a possible 600 lb. per sq. in. alteration of the mortar strength is reasonable. Observed water losses may often produce a greater change. Whatever it may be, the concrete has undergone substantially the same metamorphosis and no differential exists. Even Abrams' water-cement ratio curves have this discrepancy present. These were evolved from tests in which the equipment was no different in this respect from that now used. Were we to adopt water-tight molds, there might be detected a lowering of the Abrams' curves.

The writer fully appreciates the value of large numbers of tests, if separate, detailed stories are not lost and averaged out in the mass of accumulated data. It was to this "averaging out" that the writer alluded when he suggested the slogan "not how many, but how well."

There is much reason for believing that the water-cement ratio strength relation, based as it is on so many thousands of tests represents a broad zone that contains laws within laws. A. R. Lord ("Notes on Concrete—Wacker Drive, Chicago") gave excellent expression to this thought in his admirable paper before this convention:

> "By comparison with Abrams' general water-cement-strength equation, $S = \frac{14{,}000}{7^x}$, the test results from Wacker Drive show very high. With from 7 to 7.45 gal. of water per sack of cement our 28-day strengths ran from 3,700 to 2,900 lb. per sq. in., while the equation would give 2,300 to 2,000 lb. per sq. in. The excess is approximately 50 per cent. The temperatures of curing ran from 59 deg. to 74 deg. F. and do not account for the excess. My own conclusion from these is that the water-cement-strength relation is in fact a related series of curves varying with different cements and aggregates and with different gradations and combinations of aggregates, and covering a zone on the Abrams' graph. The particular aggregates, gradation and proportions worked out for Wacker Drive gives an Abrams' curve which lies near the upper edge of the zone and represents a highly efficient and economic use of the available material in the Chicago market."

Talbot and Richart (Bulletin 137, Uni. of Illinois) show the same thing in Figs. 9 and 44.

The standard sand 2 x 4-in. series cited early in this closure is further evidence of the same as are also the results of tests on the Joint Committee recommended mixtures for given strengths developed by Abrams. When 3,000-lb. mortars range from 3,900 to 6,300 lb. per sq. in.; 3,000-lb. concrete from 2,500 to 5,250; 2,000-lb. mortar from 2,300 to 5,050 and 2,000-lb. concrete from 1,150 to 4,300, is there not indicated a breadth and depth of zone that merits further detailed analysis? It should also be borne in mind that this disparity is largely upward and not in the weak unworkable range so much discussed.

Under his discussion of the proposed slogan "not how many but how well," Mr. Gonnerman, in a slightly condescending but kindly vein, explains: "In this connection it may be pointed out that an *understanding* of some of the *fine points* in making tests, *such as have been referred to* is *one* of the by-products of *experience* with *large numbers of tests* and in testing, as in other lines, it is the *proper* use of by-products that often spells the difference between success and failure."

The writer does not wish to appear ungrateful for this fatherly effort to "soften the blow" or "let him down easy," as it were. May he ask, however, without discourtesy or ingratitude, if the *fine points* and *by-products*,

gained from watching these many thousands of specimens pass hastily in review, are fairly represented by "*such as have been referred to.*" Are the following typical of the lessons to be learned?

1. To watch slumps but to ignore strengths and densities?
2. To watch the water ooze from the base of a mold containing a mortar specimen but to be blind to a less leakage producing an equal effect on a concrete specimen?
3. To discount and dismiss with a wave of the hand results five times checked on as many aggregates or six times checked on as many gradings because they were not all on the same aggregate or on one grading?

A bin of stone is a fine thing for some purposes but there is some knowledge to be gained from a close inspection of a few fragments in the hand or under a glass that will never be learned from dealing with binfuls.

The writer, as stated in the body of the paper, does not wish to be understood as attempting to detract one iota from the vast amount of splendid work turned out by Abrams, Gonnerman, Walker, McMillan, and others of the P. C. A. staff. Their greatest work is the water-cement ratio theory and the constructive program of education in its application. If this theory be not the whole key to the situation, the doubts raised are not for the bewilderment of the practitioner. Let him continue to believe all that is now alleged, until such time as the laboratories can offer him a further refinement in usable garb.

At the close of his discussion Mr. Gonnerman offers data that from his interpretation disagree with findings recorded in his paper. These are evidently from the series recently reported by Mr. McMillan in the *Journal* of The Western Society of Engineers and abstracted in the *Engineering News-Record*. The writer has followed these reports with interest but does not know enough of the details to attempt any discussion of the findings. The essential thing is that these very data of Gonnerman and McMillan are additional proof of divergent authority and further argument for such an investigation as the writer is advocating.

On the one hand we have a strong and strenuously defended contention that the alpha and omega of concrete lie in the water-cement ratio relation. No investigation of aggregates is needed or desired. The P. C. A. Laboratory is the leading exponent of this viewpoint. They are not entirely alone, however. R. B. Young, eminent and disinterested investigator, verbally assured the writer that he had test data demonstrating the adequacy of the water-cement ratio theory.

On the other hand we have such authorities as Talbot, Richart, Lord, Williams, Davis, Hutchinson and Wheeler producing evidence that there are still important things to be learned about the aggregate in its relationship to the properties of the concrete.

The need for an investigation has surely been established. Whether launched or not, it is hoped that some few may find in this paper and the accompanying discussions new stimuli for constructive thought and action.

A New Test Method for Workability of Concrete.

By Tokujiro Yoshida.*

I. Introduction.

1. *Workability of Concrete.*—The quality of concrete or mortar which has to do with ease of handling and placing may be referred to as "consistency," "mobility," "flowability," "plasticity," or "workability." Some authorities prefer to distinguish between the words workability and consistency, considering the latter as a property directly related to the water content, whereas workability is regarded more intrinsically as characteristic of a given mixture, depending not only on the water content, but essentially upon the other ingredients of the mixture. Although these words have perhaps slightly different meanings to different people, they are generally considered synonymous.

2. *Importance of Workability on the Properties of Concrete.*—Concrete to be used in construction must not only have sufficient strength to meet the stress it is called upon to carry, but must be sufficiently mobile or workable, in order that the operations, such as properly filling the forms or completely embedding the reinforcement, may easily and safely be executed, with reasonable care. There is some minimum workability which must be obtained as well as minimum compressive strength for the safety of structures, in order to result in good workmanship, and economy and proper progress in the process of concreting, depending on the nature of the work, type and arrangement of forms and reinforcing steel. Therefore, strength and workability are two fundamental requirements for the design of concrete mixtures for any given condition. Any variation in the proportions, the mixing water, characteristics of the aggregates, the method of manipulation, etc., is reflected at once in the resulting strength and workability of the concrete. Hence concretes are only comparable when their strengths and workabilities are given. Even with given materials and proportions, by the use of more mixing water alone, workability may be increased, and this in turn will lower the compressive strength. The strength of concrete can be correctly determined by a testing machine; the correct determination of workability, then, becomes a condition precedent to the proper testing of concrete and absolutely the most important feature of concrete investigations. The effect of other variables on the quality of concrete may be entirely masked by what may appear to be slight variations in workability.

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3. *Difficulties in Workability Measurement of Concrete.*—The workability of concrete may be governed by the following factors:

1. The normal consistency of the cement;
2. The size and grading of the aggregate;
3. The absorption of the aggregate;
4. The quantity of water contained in the aggregate;
5. The relative volumes of cement and aggregate;
6. The quantity of water;
7. The time of mixing;
8. The method of handling;
9. The time elapsed after mixing.

While it will not be feasible to discuss each of the foregoing factors at length, it is apparent that a most complex relation is involved.

It is considered that one concrete mixture is more workable than another when the process of mixing, handling and placing are accomplished with less effort in one case than in the other. But direct quantitative measurement of this effort, for example, in placing a concrete mixture in any sort of a form or mold is one of the difficulties, chiefly because there is no telling when one has arrived at a definite condition that may be taken as a stopping point in the placing operation. Also the time elapsed after mixing sometimes gives a marked difference in workability due to settlement or segregation of materials, especially when the mixture is rather coarse or very wet.

Granting a certain condition of handling and timing after mixing, there are other fundamental difficulties in the way of the development of a simple yet reliable method for determining the workability of concrete. In general, the workability of concrete varies from that of moist earth to that of viscous liquid, the intermediate conditions being most commonly met with. If concrete is in the moist earth condition, as is used in road construction, it is considered that the workability may be measured by the work done to compact the concrete to a certain density. If concrete is plastic, the workability may be expressed by the ratio between the rate of flow and the force producing this flow, which it is probably quite impossible to express completely by any single numerical value, for the reason that the ratio for plastic material is different at low and at high rates of flow. If concrete were a viscous liquid, a single numerical value of viscosity would express its workability. As a matter of fact, concrete is neither a perfectly plastic material nor a perfect liquid, and the presence of aggregate serves to complicate the problem greatly. It is only a rough approximation to assume that the concrete is homogeneous when the maximum size of the aggregate is at all large compared with the volume of concrete which is generally used for the workability measurement.

From the foregoing discussion, it will be seen that a correct determination of workability of concrete by a simple device is quite impossible. However, necessity, the mother of invention, has suggested several important

methods for approximately determining this factor, and it is hoped that the drop test method herein described may be one of them.

4. *Methods of Measuring the Workability of Concrete.*—The following methods have been tried at various times:

1. Measurement of angle of repose (By H. A. Thomas);
2. Rate of flow down chutes;
3. Measurement of lateral pressure (By Prof. A. N. Talbot);
4. Flow through orifices;
5. Measurement of rise of concrete at the side of a cylinder by centrifugal force when the cylinder is revolved at a certain speed (By W. L. Schwalbe);
6. Measurement of resistance to moving steel rods passing through concrete;
7. Slump test;
8. Flow table;
9. Penetration test (By J. C. Pearson and F. A. Hitchcock);
10. Plate device (By F. H. Jackson and George Werner);
11. Resistance to extraction of a cone embedded in concrete (By the writer. See Fig. 1);
12. The apparatus shown in Fig. 2 (By the writer).

Of these methods, only two or three have ever been developed so as to be of any practical value.

The slump test has been used more extensively than other methods as a method of field control. It is characteristic of the workability measurement by this method that marked differences in slump are produced by the difference in quantity of mixing water over a very narrow range only, and the limitations, especially as regards its use in the control of dry mixtures, lean mixtures and concretes in which large-sized stone is used, have prevented its general adoption. Another thing to contend with in the slump test is that two successive measurements are likely to differ widely, even though the concrete is the same in both cases, and it is impracticable to make tests frequently enough to have several values for obtaining an average at a given time.

The flow test has also been tried out by a number of investigators and is used to a certain extent as a method of laboratory control. No doubt this is one of the good solutions for determining the workability of concrete. However, in the writer's experience it seems to be open to the following criticisms. The chief criticism of the flow table method, like some other methods, is that it is concerned with determining the workability of concrete in a segregated condition not as a whole mass when the mix is rather coarse or reasonably wet. With such mixtures, when the mold is taken away after it is filled, the mortar separates from the coarse aggregates, cement paste from the mortar, and water and laitance from the paste. In such a case the effect of a certain number of drops only accelerates the flow of the paste and water, and the diameter to be measured to

determine the flowability is open to question. If the diameter of the "flow" of the cement paste or water is measured, the "flow" is proportional to the mixing water, which is proposed as a characteristic of this method. But it only shows the quantity of water used, and does not show the workability of the concrete as a whole. Hence, with the flow table reliable results may be obtained only with the concrete in which practically no segregation occurs during the process of the test. However, the concrete generally used, as for reinforced-concrete construction, has a tendency to segregation during the process of the test. Hence this method is not considered reliable for comparing the workabilities of concretes.

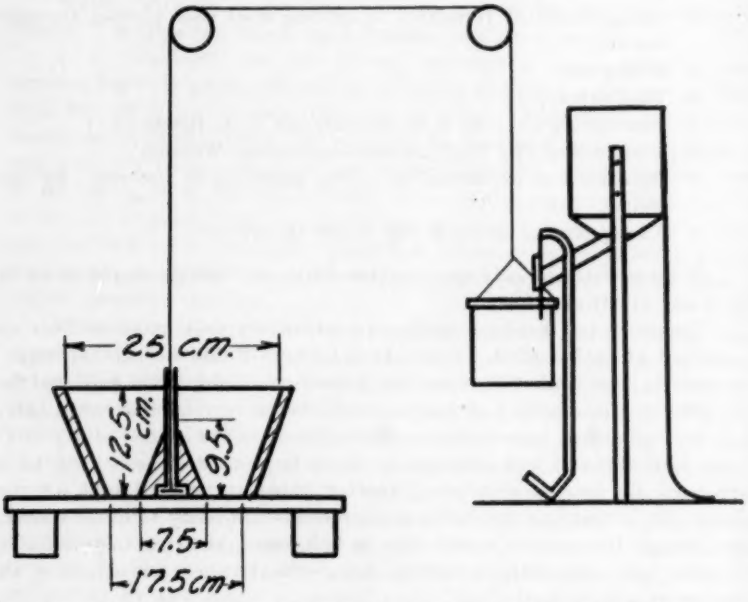

FIG. 1.—WORKABILITY DETERMINED BY PENETRATION TEST DEVICE.

The penetration test for the workability measurement of concrete was, no doubt, suggested by the use of the Vicat needle for determining the normal consistency of portland cement. The method was tried in order to measure the tendency of a mixture to pack under a definite amount of settling and compacting. It seems to give good results only when the segregation of the material is comparatively small and is lacking in desirable precision.

The plate device is proposed for concrete used in road construction and it is clear that this is not suitable for the concrete either in a plastic or in semi-fluid condition.

The method shown in Fig. 1 was tried by the writer in applying the principle of the penetration test. Instead of penetrating a rod into concrete the resistance offered in pulling out a wood cone from concrete was measured. The method seems to give a little better result than the penetration test, but has approximately the same defects.

The apparatus shown in Fig. 2 is a kind of viscosity meter. It is really the reverse of the slump test. Instead of pulling up the cylinder, the concrete in the cylinder was pushed upward from the cylinder, and the

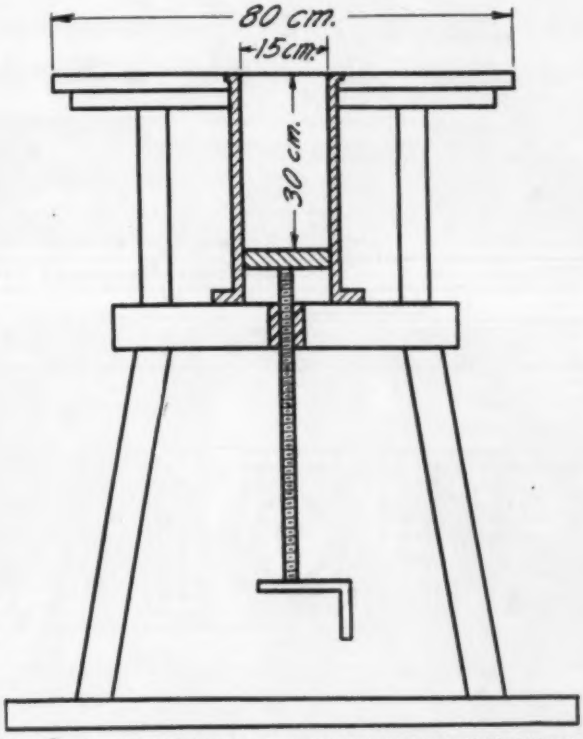

FIG. 2.—VISCOSITY METER FOR DETERMINING CONSISTENCY.

diameter of the flow was measured. This gives approximately the same result as the flow table. The chief criticism in this case is the same as in the flow table, i. e., it measures the workability of concrete in the segregated condition only when it is rather coarse or reasonably wet.

5. *New Device for Measuring the Workability of Concrete.*—To overcome the defects of the methods of the workability measurement of concrete referred to in the foregoing discussion, a new device is proposed. This is the first worthwhile solution of the problem which the writer has

found in a research extending over a period of several years. The process is based upon the principle that the workability of concrete may be determined by measuring the diameter of the concrete upon a plate when the concrete is dropped thereon in a standard manner. The device which has been used in demonstrating this principle is shown in Fig. 3.

The apparatus consists essentially of a metal mold which has the shape of a hollow frustum of a cone, supported by a frame over a plate at such a height that the point of discharge is 20 cm. from the surface of the plate.

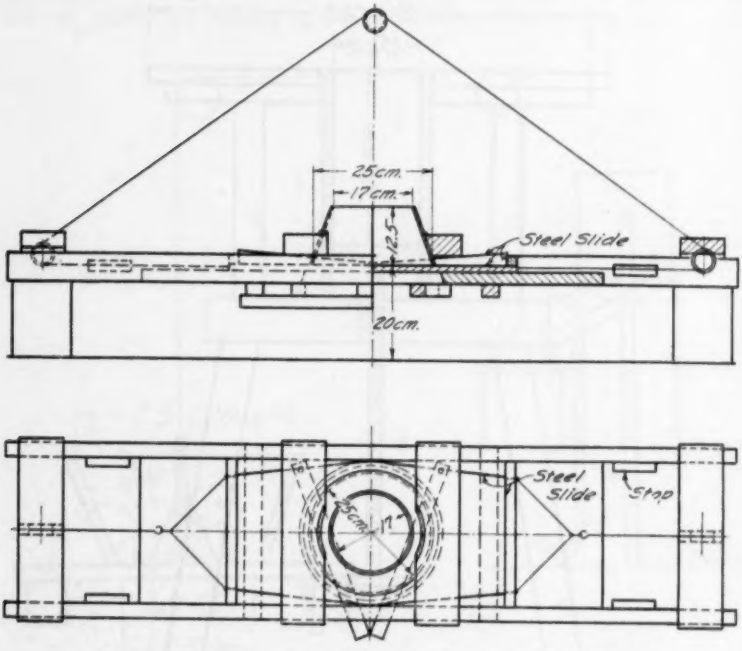

FIG. 3.—DROP TEST DEVICE FOR DETERMINING WORKABILITY.

The mold is provided at the discharge end with two steel slides which meet at the middle. For aggregates up to 3.75 cm. maximum size this truncated cone has a height of 12.5 cm. and upper and lower diameters of 17 and 25 cm., respectively.* To make the mold water-tight for wet mixtures, it is sometimes necessary to seal the contacts of the steel slides and also of the bottom edge of the cone and steel slides with pomade.

The test is made in the following manner as shown in Fig. 4 and Fig. 5. The mold is filled about one-half of its height with the concrete

* This corresponds closely with the dimensions of mold used in the standard flow test. (Standard Methods of Making Compression Tests of Concrete, C39-25, A. S. T. M. Standards.)

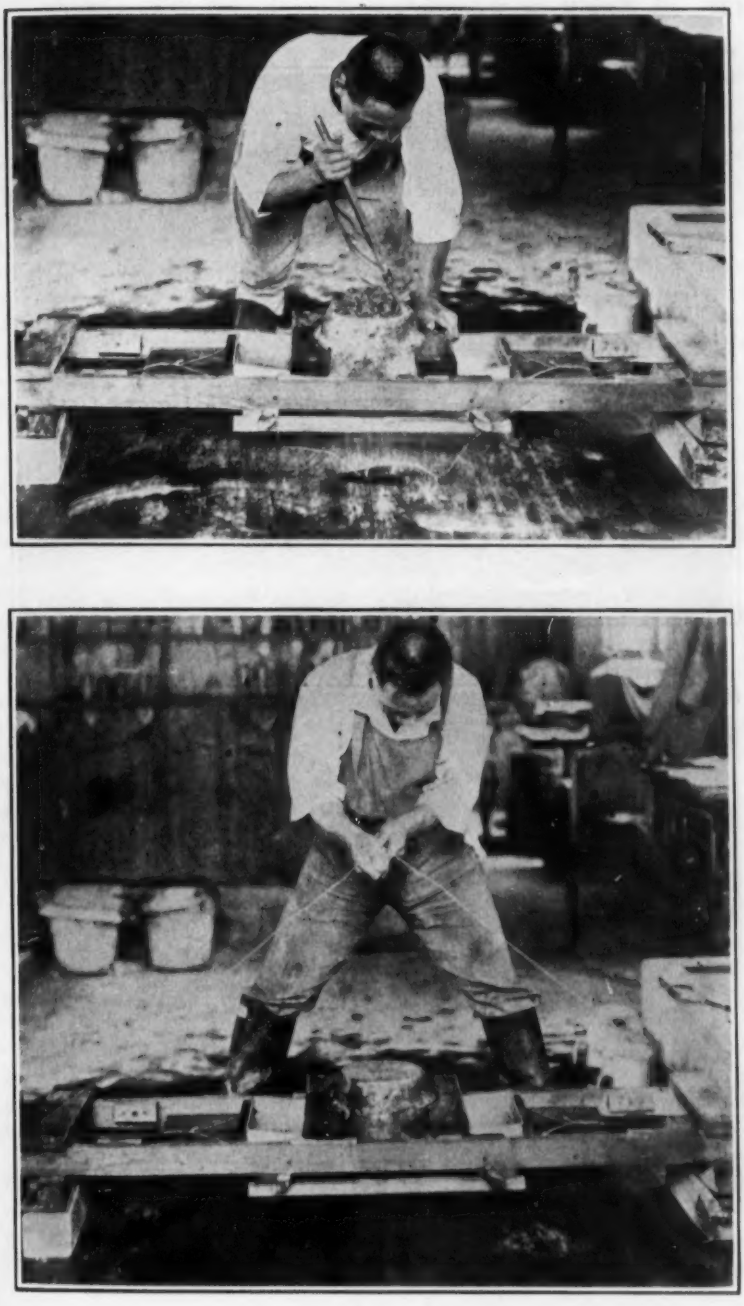

FIG. 4.—DROP TEST DEVICE FOR DETERMINING WORKABILITY.

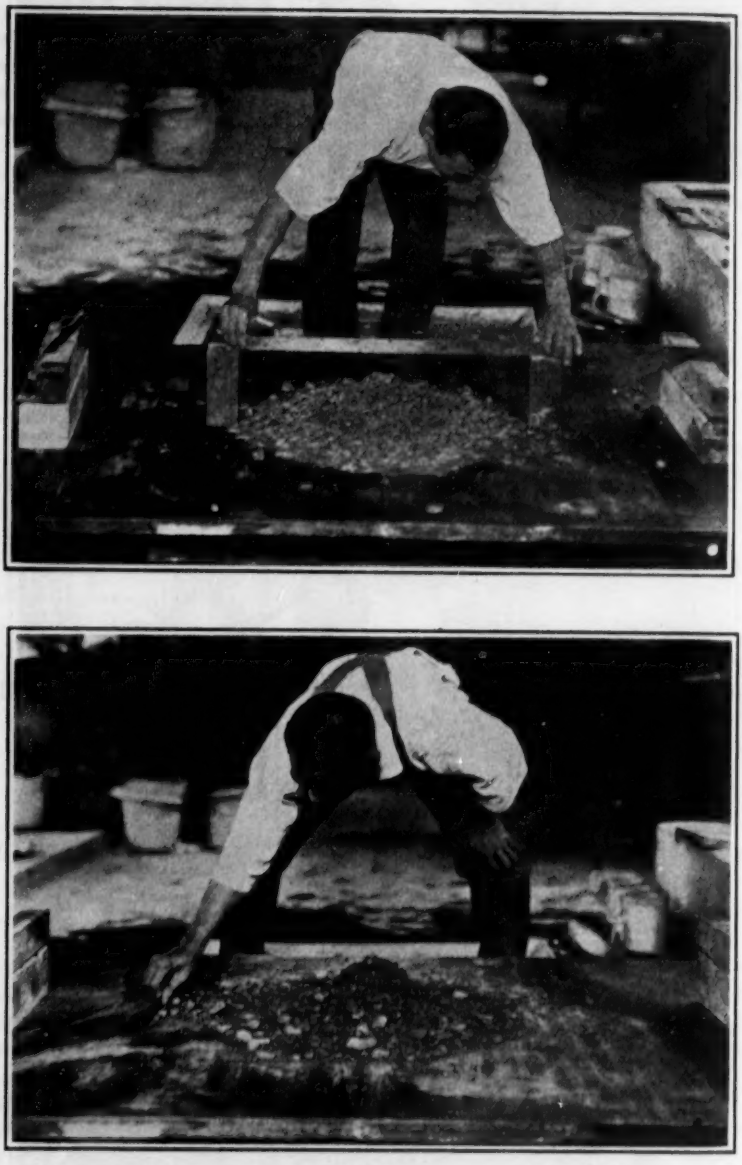

FIG. 5.—CONCLUDING DROP TEST FOR MEASURING WORKABILITY.

which is then puddled, using 25 to 30 strokes of a 15 mm. rod pointed at the lower end. The filling is completed in successive layers similar to the first and the top struck off, so that the mold is exactly filled. When the mold is filled, the steel slides at the bottom of the cone are drawn away in opposite directions by pulling the wire rope which is connected with the steel slides at its ends through pulleys, and the concrete is allowed to drop through a distance of 20 cm. upon a plain wood plate. The concrete mass flattens and spreads concentrically. Four diameters at 45 deg. to each other are measured. The mean of these values divided by the lower diameter of the cone is proposed to express the value of "workability." When the mixture is too dry or too coarse so that the coarse aggregates spread out from the mass, particles may readily be brought within the border of the mass by a steel straightedge as shown in Fig. 5, and the diameters measured. When the mixture is such that the water and cement paste flow away from the mass, it is necessary to set some marks at the edges of the concrete at the instant of drop, and the diameters are measured by the distances of the marks. It is considered a characteristic of this device that the workability of the concrete is measured as a whole mass, as the diameters at the instant of drop are measured, i. e., not in the segregated condition of water, cement paste and aggregates. By this device, the concrete tested is subjected to treatment similar to that in depositing it by shoveling and chuting and the result of the test no doubt gives some measure of the workability of concrete.

Numerous tests with this device hereafter shown on concrete of various proportions and with various sizes and gradation of aggregates indicate that it will accurately measure the relative workability of concrete or mortar for all workabilities varying from a dry mass to those workabilities which are so fluid that water and laitance will flow away from the coarse aggregates. As the device is simple, it is suitable for the field control of concrete.

II. Experiments on the Workability of Concrete With the Drop Device.

6. *Materials.*—The materials used were similar to the best materials used for concrete in the Fukuoka district.

Cement: Onoda portland cement used was furnished by the maker.

Sand: The sand from the Nakagawa River was of good quality, well graded, fairly clean, containing 36.9 per cent voids and weighed 1,635 kg. per cu. m. The results of the mechanical analyses of the sand are given in Table I.

Gravel: The gravel was from the Moromi River, the chief constituents being hornblend and granite. It was passed through a 1-in. screen and over a ¼-in. screen; it contained 38.5 per cent voids and weighed 1,693 kg. per cu. m. Table I gives the average of several mechanical analyses of the gravel. In order to obtain comparable results in the series of tests, it

was felt necessary to use the same grading of coarse aggregate in the mixtures. The gravels were screened through ¾, ½, and ¼-in. sieves and recombined in the proportions shown in Table I.

Concrete: An effort was made to have the condition of fabrication of the different concrete mixtures as nearly uniform as possible. The mixing was done by hand with shovels. The sand and cement were first mixed dry, the water then added and the mass turned over; the gravel was then added and the whole mass turned over until thoroughly mixed. The mixture was spread out flat in the mixing pan and the center part of it was taken out by small shovels to fill the mold. To get a check test on the quality of the concrete a number of 15 x 30-cm. cylinders were made from the batch.

TABLE I.—SIEVE ANALYSIS OF SAND AND GRAVEL.
(Tyler Standard Sieves Used.)

Material	Sieve Size	Size of Square Openings, In.	Percentage Retained
Sand	No. 100	0.0058	99.6
	No. 48	0.0116	98.1
	No. 28	0.0231	84.8
	No. 14	0.0460	54.1
	No. 8	0.093	14.0
	No. 4	0.185	3.4
Gravel	No. 4	0.185	99.6
	No. ⅜ in.	0.371	80.9
	No. ¾ in.	0.742	11.7
	No. 1 in.	1.050	0.0

7. *Workability of Cement Paste and Mortar.*—Fig. 6 shows the workability of 1: 1, 1: 1¼, 1: 1.54, 1: 2, and 1: 3 volume proportion mortars tested with the drop device, using zero to 90 per cent mixing water to weight of cement.

The curve designated as sand shows the workability of the sand alone with various percentages of water by weight. This is considered the numerical representation of the phenomenon that is exemplified by the penetration of one's foot in the sand when walking along the seashore. The sand, which is very mobile when it is dry, becomes plastic as the moisture content increases and reaches its least workable condition at about 25 per cent water content to weight of the sand. When the water content increases over that amount the workability also increases in about direct proportion to the water content.

The curves for the workability of mortars show that the mix is mobile when no mixing water is added as in the case of dry sand, and has a high workability value. When some water is added, the workability decreases rapidly due to the cohesive and adhesive powers of the thick cement paste and reaches its minimum value at a certain water-cement ratio. When some water is added, the lubrication of the cement paste between the par-

ticles of sand comes into action and the workability increases in about direct proportion to the water-cement ratio.

The curve of workability for cement paste practically conformed to the curve of 1 : 1 mortar and is not shown in Fig. 6.

The cement paste or mortar having the minimum value of workability corresponds nearly to the paste or mortar of maximum density and strength for the mix, assuming the ordinary condition of depositing or tamping.

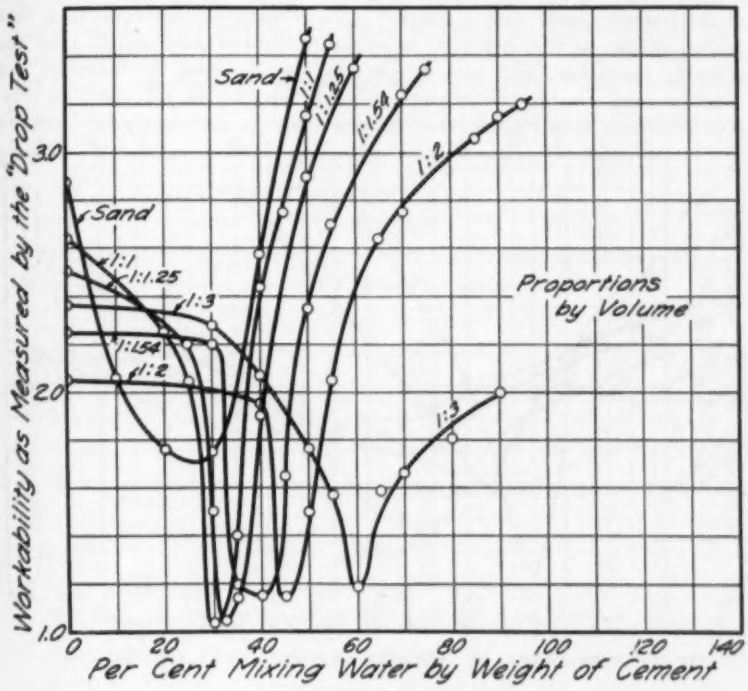

FIG. 6.—WORKABILITY OF MORTAR AS MEASURED BY THE DROP TEST.

8. *Workability of Concrete of Various Proportions and Quantities of Mixing Water.*—Typical curves showing the relation between the workability of concrete as measured by the drop device and the quantity of mixing water for six separate batches of concrete of 1 : 1 : 2, 1 : 1½ : 3, 1 : 1¾ : 3½, 1 : 2 : 4, 1 : 2½ : 5 and 1 : 3 : 6 proportions by volume with various percentages of mixing water to weight of cement, are shown in Fig. 7.

When no mixing water is used the mix is as mobile as in the case of dry sand, and has high value of workability. When some water is added the workability decreases due to the adhesive power of the thick paste and reaches its minimum value at a certain water-cement ratio.

When more water is added the lubrication of cement paste or water between the particles of aggregate comes into action and the workability increases in about direct proportion to the water-cement ratio as in the case of mortar.

For a given proportion of cement and aggregate, concrete of minimum workability contains about a sufficient quantity of mixing water to produce concrete of maximum density and strength for the ordinary field condition for distribution and depositing concrete. The density and strength of the concrete made with a smaller value of water-cement ratio than that corresponding to the minimum workability depend very much upon the tamping operation; and such a concrete may be used, for example, for concrete road construction where thorough tamping is possible. In reinforced-concrete construction the concrete must be sufficiently workable to

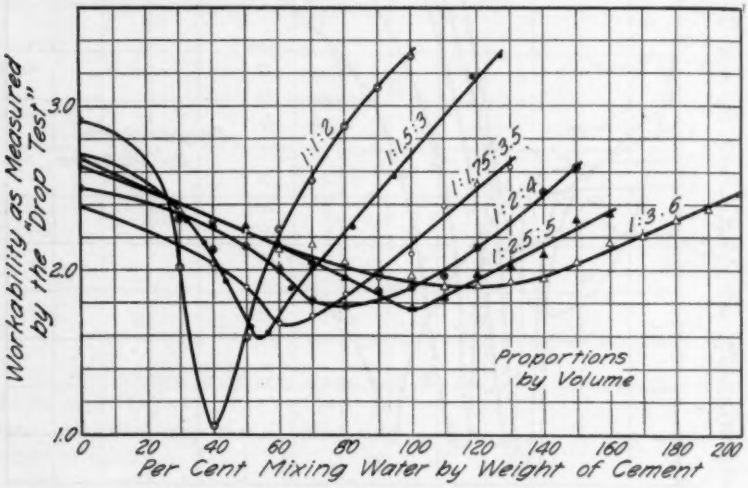

FIG. 7.—WORKABILITY OF CONCRETE AS MEASURED BY THE DROP TEST.

allow of the operations of properly filling the forms and completely embedding the reinforcement being easily and safely executed, with reasonable care. In such a case the concrete must be made with a greater value of water-cement ratio than that corresponding to the minimum workability of the mix.

As to the relation of the workability and the mix, the richer the mix, the greater is the variation of workability due to the variation of water-cement ratio, and the less is the value of the minimum workability and the water-cement ratio corresponding to it. Therefore, with a rich mix it is possible to increase the workability by a little increase of the value of the water-cement ratio, and hence without much decreasing the strength of the concrete. For the same workability, the richer the mix the less is the value of the water-cement ratio and hence the stronger the concrete.

With a lean mix, when the water-cement ratio is less than that corresponding to the minimum workability the volume of cement paste is not sufficient to coat the surfaces of the aggregates and to fill the voids. Hence the adhesive power of the thick cement paste between the particles of aggregates can not fully develop, thus producing a harsh mixture. At the water-cement ratio producing a sufficient volume of cement paste to develop its adhesive power the workability reaches its minimum value. As that water-cement ratio is greater than that for a richer mix, the cement paste being thinner, the value of minimum workability is higher than that of a richer mix. When the water-cement ratio increases from that point, the tendency of the mixture to segregate also increases and

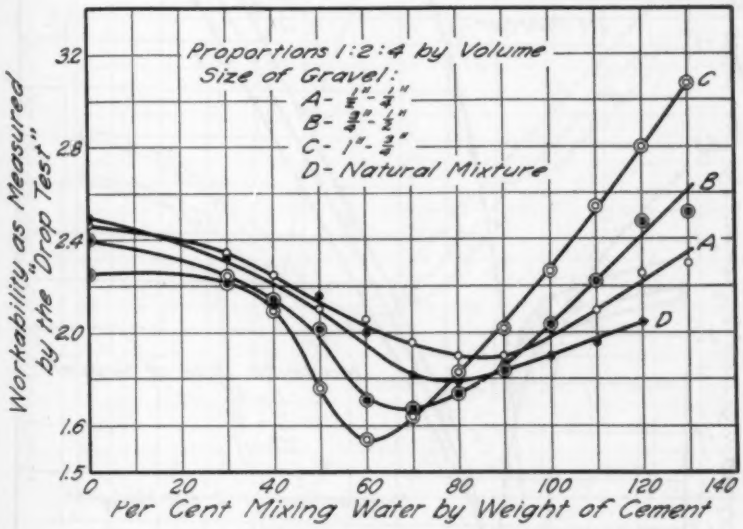

FIG. 8.—EFFECT OF SIZE AND GRADATION OF COARSE AGGREGATE ON WORKABILITY OF CONCRETE.

in extreme cases, as when water is added to gravel, the workability does not increase.

From these curves it will also be seen that the workability depends more upon the proportions of solid ingredients in the concrete than upon the amount of mixing water used.

9. *Effect of Size and Gradation of Coarse Aggregates on the Workability of Concrete.*—To obtain some idea of the effect of size and gradation of coarse aggregates on the workability of concrete, the following tests were made:

1: 2: 4 volume proportion concrete using three different sizes of gravel, A, B, and C, as coarse aggregates with various percentages of mixing water to weight of cement were tested by the drop device.

A—passing ½-in. sieve and retained on ¼-in. sieve;
B—passing ¾-in. sieve and retained on ½-in. sieve;
C—passing 1-in. sieve and retained on ¾-in. sieve.

The result of the test is shown in Fig. 8.

From the figure, it will be seen that the larger the size of the coarse aggregate, the less is the value of the water-cement ratio producing the concrete of minimum workability, and the less is the value of the minimum workability. This is due to the fact that for coarse aggregate of larger size the surface to be coated by mortar or cement paste is less than that

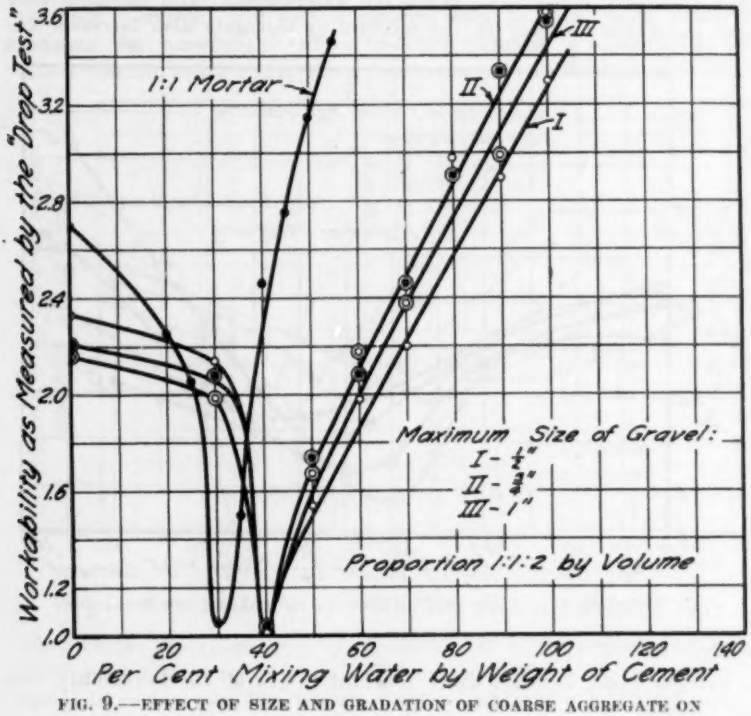

FIG. 9.—EFFECT OF SIZE AND GRADATION OF COARSE AGGREGATE ON WORKABILITY OF 1 : 1 : 2 CONCRETE.

of smaller size and hence a less volume of thick cement paste is required to produce concrete of minimum workability.

Curve D shows the workability of 1 : 2 : 4 volume proportion concrete using as aggregate the natural gravel shown in Fig. 7. It lies between curves A and B until it reaches the value of the water-cement ratio corresponding to the minimum workability and beyond that a little under curve A. As the natural gravel has less voids than those that have been separated, the segregation of materials in the concrete with comparatively high water-cement ratio is less, and the workability of the concrete is

relatively low. This shows that the control of the grading of the coarse aggregate for concrete deserves the very best consideration and study, not only for the sake of the strength, but also for the workability of the concrete.

Figs. 9 to 12 show the workabilities of four separate batches of concrete of 1: 1: 2, 1: 1½: 3, 1: 2: 4 and 1: 3: 6 proportions by volume with

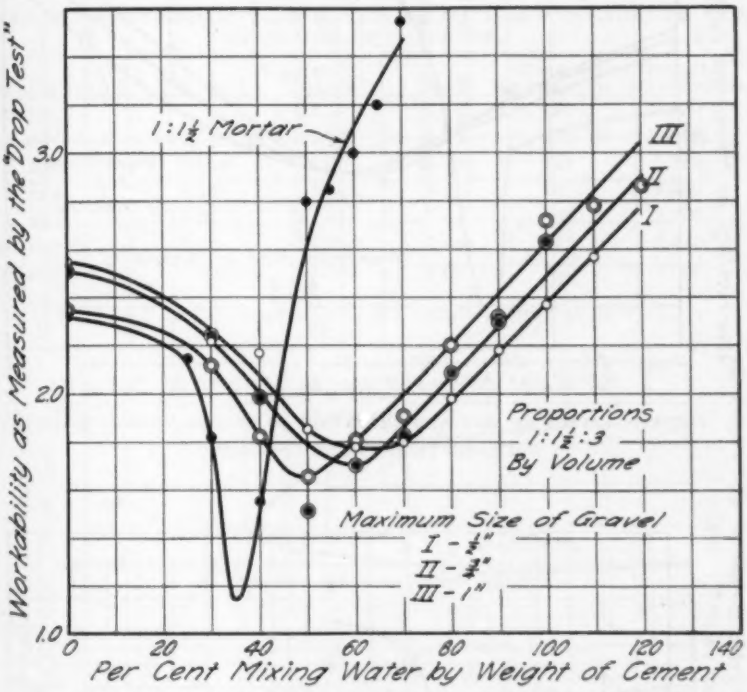

FIG. 10.—EFFECT OF SIZE AND GRADATION OF COARSE AGGREGATE ON WORKABILITY OF 1: 1½: 3 CONCRETE.

varying percentages of mixing water, using coarse aggregates of three different maximum sizes, namely:

I. Gravel A;

II. Gravel A and B combined to produce maximum density (7: 3 by weight);

III. Gravel A, B and C combined to produce maximum density (1: 2.3: 7.7 by weight).

From these figures it will be seen that the relations between the workability of concrete, proportion of cement and aggregate, water-cement ratio, the value of water-cement ratio corresponding to minimum workability, and the value of minimum workability are the same as in Fig. 7,

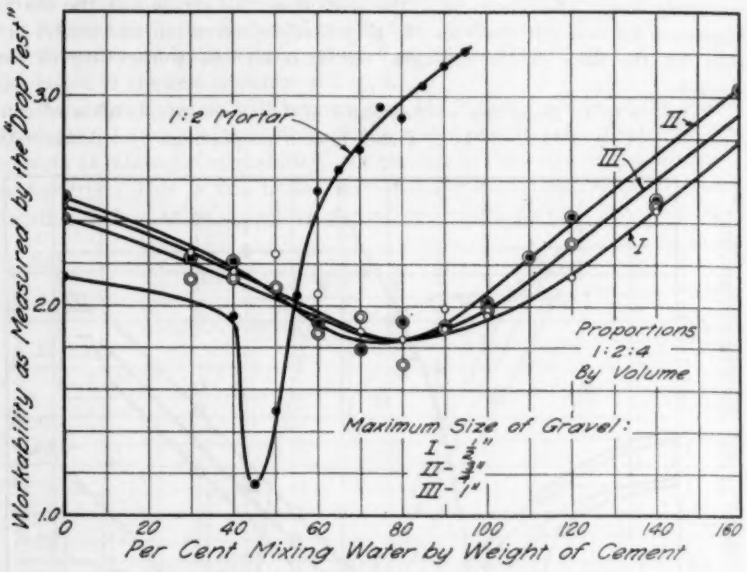

FIG. 11.—EFFECT OF SIZE AND GRADATION OF COARSE AGGREGATE ON WORKABILITY OF 1: 2: 4 CONCRETE.

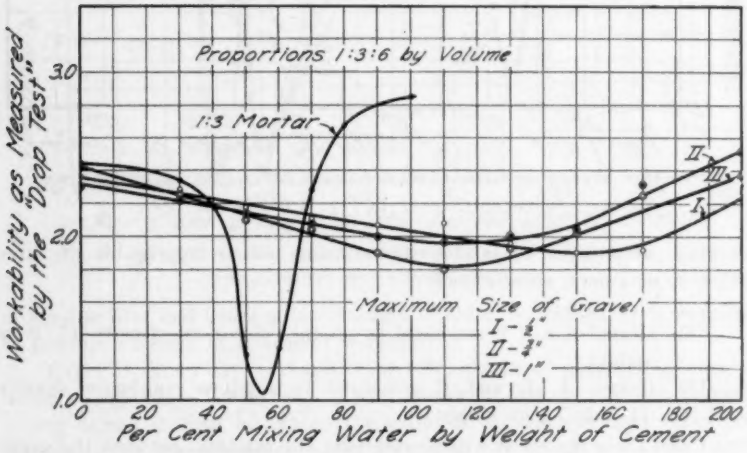

FIG. 12.—EFFECT OF SIZE AND GRADATION OF COARSE AGGREGATE ON WORKABILITY OF 1: 3: 6 CONCRETE.

and that the maximum size of coarse aggregate, when the aggregate is of the gradation of maximum density, has not much effect on the workability of concrete.

10. *Effect of Time of Mixing on the Workability and Strength of Concrete.*—The effect of the duration of the mixing operation on the workability and strength is shown in Table II.

TABLE II.—EFFECT OF TIME OF MIXING ON THE WORKABILITY AND STRENGTH OF CONCRETE.

Time of Mixing	Workability			Compressive strength*	
	Test No. 1	Test No. 2	Mean	Kg. per sq. in.	Lb. per sq. in.
30 sec.	....	1.75	1.75	93	1,320
1 min.	1.775	1.735	1.76	112	1,590
2 min.	1.755	1.64	1.70	121	1,720
3 min.	1.70	1.73	1.72	137	1,950
5 min.	1.67	1.61	1.64	162	2,300
10 min.	1.55	1.604	1.58	172	2,440
15 min.	1.601	1.595	1.60	176	2,500

* 15 x 30-cm. (approximately 6 x 12-in.) cylinders tested at 28 days.

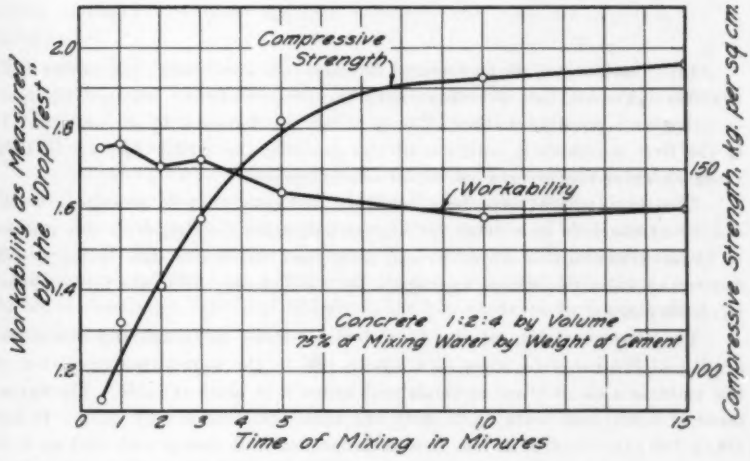

FIG. 13.—EFFECT OF TIME OF MIXING ON WORKABILITY AND STRENGTH OF CONCRETE.

Fig. 13 is the diagrammatic representation of the mean values shown in Table II. Concrete of a 1: 2: 4 mix was used in the test with 75 per cent of mixing water by weight of cement.

The concrete was mixed in a drum mixer (capacity 4 cu. ft., 22 revolutions per minute) manufactured by the Ajigawa Iron Works, Osaka. The mixed concrete was discharged into a pan and turned over twice with

shovels and spread evenly in the pan. The workability of the concrete was tested by the drop device. Fifteen by thirty-cm. cylinders for compressive strength were made from the batch and tested at the age of 28 days.

From Fig. 13 it will be seen that the time of mixing has not much effect on the workability of concrete, as far as the usual mixing period is concerned. It may seem unreasonable that the workability has a tendency to decrease when the time of mixing is over three minutes, but it was actually observed during the test that concrete mixed in the shorter time appeared more workable and also handled easier. It is probably due to the fact that the surface of the cement in contact with water at first, was broken by the longer mixing, and a new surface came into contact with water, thus producing a thicker cement paste and hence decreasing the workability and making a stronger and more uniform concrete. The grinding action on the aggregates in the longer time of mixing may also influence the workability to some extent. As the quantity of water absorbed by the aggregate used is very small, the stiffening of the mass of concrete by absorption during mixing needs no consideration.

From this test, it may be concluded that a longer time of mixing is important in obtaining stronger and more uniform concrete, but does not much affect its workability.

III. CONCLUSION.

11. *Summary of Experimental Results.*—In conclusion, the writer feels that enough work has been done with the drop device to warrant its presentation as a possible method of control for the workability of concrete. It is the first worthwhile solution of the problem the writer has found in a research extending over a period of several years.

The data which have been obtained indicate that it measures workability of concrete or mortar for all workabilities varying from dry masses to those workabilities which are so fluid that the water and laitance will flow away from the coarse aggregate, and sufficiently accurate for all practical purposes.

The characteristics of the drop device consist in measuring the workability of the concrete mass as a whole, not in the segregated condition of the materials as in other methods and hence it is more reliable. The agreement of individual tests made with the same concrete is very good. It has about the same merits as the flow table method and slump test, and as it is simple, it may be used for the field control of concrete.

Another advantage of this method is that the quantity of mixing water to produce concrete of maximum density and strength for the ordinary field condition of depositing in a given mix can easily be estimated by the test.

It is believed that the following conclusions are justified as to the workability of concrete tested by the drop device:

(1) The workability of concrete or mortar decreases as the water-cement ratio increases from zero, and reaches its minimum value at a

certain water-cement ratio. After that the workability increases in about direct proportion to the water-cement ratio.

(2) The richer the mix, the greater is the variation of the workability due to the variation of the water-cement ratio, and the less is the value of the minimum workability and the water-cement ratio corresponding to it. Workability depends more on the proportions of solid ingredients in the concrete than the amount of mixing water.

(3) For a given proportion of cement and aggregate, concrete of minimum workability contains about a sufficient quantity of mixing water to produce concrete of maximum density and strength for ordinary field conditions of distributing and depositing concrete.

(4) The maximum size of coarse aggregate, if the aggregate is of a gradation to give maximum density, has not much effect on the workability of concrete.

(5) For coarse aggregate of nearly uniform size, the workability of the concrete is greater for the smaller sizes than for the larger ones when the water-cement ratio is less than that corresponding to the minimum workability, and is less when the water-cement ratio exceeds that value.

(6) Time of mixing has not much effect on the workability of concrete as regards the usual duration of mixing. Longer mixing is important in obtaining stronger and more uniform concrete, but does not increase its workability.

The foregoing description of the drop device and its method of use is given with the hope that those who are interested in the testing of concrete will give the method a trial, and any criticism based upon its application will be appreciated.

12. *Acknowledgment.*—The tests were made in the concrete laboratory of the civil engineering department of the Kyushu Imperial University. The writer desires to acknowledge the valuable assistance of M. Kawaguchi, R. Aoki, and D. Shirozu in carrying out the work of this investigation. It is also desired to make an acknowledgment to Hikoichi Motoyama, the president of the Osaka Mainichi Shimbun, for his kindness in contributing to the expenses of the test.

Time as a Factor in Making Concrete.

By M. M. Upson.*

We approach now the theme which is to engage our attention for the last two sessions. It is a new theme and an extremely interesting one. It is also a little bit difficult of presentation, and in view of that I have taken the liberty of introducing it in a written form, believing that by so doing I will save time and make the statement in a more lucid way than I would in addressing you extemporaneously. The very substance of an academic society like the American Concrete Institute is fellowship. Fellowship is the study of a subject of common interest. With us that subject is concrete. It presents as many aspects as there are points of view, but does not lose its identity. Like a beautiful jewel, the facets are many but the substance is one. As one facet of a jewel reflects light more completely than the others, so one phase of concrete attracts more perfectly than the others the attention and interest of some members or group of members. The Institute is conscious of this tendency on the part of its members, and in presenting from year to year the best that is available in thought and practice, it strives to hold the interest of the members as firmly as possible to the principles underlying the whole subject. At the same time it wishes each member to receive what is of particular interest to him, what will help him most in his work.

The program committee has, therefore, arranged a program for this convention including sessions of particular interest to each group of members. It has, in addition, prepared for these two last sessions a theme which is of the essence of the making of concrete, which is as important to each member as it is to the whole society, which if understood and put into practice can only result in the making of better concrete. The title of our theme is "Time as a Factor in Making Concrete." The theme is unusual. Its presentation is unusual. I think I may safely say that the concept of time is one of the most difficult in natural philosophy. It is something we know but do not often express. It is something existing within us and without us. We agree very well about our internal representation of time. We agree to the idea of succession. We distinguish between the past, present and future, and we know that these parts cannot exist at the same time but must follow one another. Again, we agree to continuity; we cannot imagine a moment however short in which we do not grow older, nor the failure of one moment to follow another immediately.

This last conception of time is the one which applies to the making of concrete. By it separate and successive physical states of materials are related in our minds as past, present and future are related. By it control is exercised through manipulation over materials inducing separate and successive states designed to progress in order to a predetermined state of considerable endurance. When we find materials in the earth, when we gather them and crush them, when we mix them or wet them or place

* President, American Concrete Institute.

them in molds, we are inducing separate states of these materials, and if these states be related in ordinary sequence, the relation is time. When the materials for concrete are first mixed and then wetted, the relation is time. When a batch of concrete is made in a mixer and then placed in the form, that relation is time. The wetted concrete is represented in two states; in the mixer it is free, in the forms it is confined and the relation of these two states is under our control, we can lengthen or shorten the duration of one or the other; therefore time is a factor in the making of concrete.

There are many processes in which time is a factor. There comes a step in a manipulation which must be delayed until something has happened which in some measure is independent of the manipulator—a change, a development of the material within itself. The potter must pause in the manipulation of his material until time has worked its part between clay and water, a part which is the final factor in reducing the clay to a condition in which he may put it on the wheel and it will become responsive to his touch. This same factor of time is evidenced in the daily work of the housewife; she recognizes that time must be a factor in the raising of her dough; she recognizes that time is a factor in applying heat to that dough and that time is a factor in the degree in which this heat must be applied.

Perhaps the best illustration of time is that which comes to us in considering the egg. An egg with 30 B.t.u.'s applied to it in 3 min. becomes hard boiled; but with the 30 B.t.u.'s applied to it in 3 weeks, it becomes a living chicken.

This is exactly the idea that we want to get through the minds of those who are manipulating this very delicate matter, of the relationship of time in concrete. We have learned how important is water in portland cement mixtures, important as a chemical agent, important as a vehicle, important as a means of manipulation. We have learned that a certain optimum ratio of water to cement is important to the best results. We shall here today inquire whether that ratio of water has its powerful influence upon the resulting concrete at the time of mixing or at the time when the mix begins to harden. If we may use enough water as a vehicle to facilitate placing and then take away the excess water, leaving the ideal water cement ratio for the best permanent results, this is very important and we will be glad to know just how the principles may be taken advantage of in the various applications of concrete. Since it is fundamental to the entire operation, we must know something of the characteristics of cement. We must know how cement acts in hardening and how its constituents act when their re-action to water reaches that critical stage when conditions influence the final result. No one is better able to tell us than a member of this Institute, without doubt this country's foremost cement chemist, who is an acknowledged authority not only on cement technology, but who has for the better understanding of its phenomenon a very deep knowledge of ceramics, the realm of the potter already referred to. I am glad to introduce Dr. P. H. Bates, of the U. S. Bureau of Standards.

Why Time is a Factor in the Study and Use of Cement.

By P. H. Bates.*

Cements have a unique position in the construction industry. Steel, wood, stone, brick, tile, etc., are sold as having certain physical properties fully developed at the time they are placed in the structure. Cements are sold as being capable of developing the wished-for property at some time, after they have been actually used. Within the limits of error of the equipment and methods used it can be determined what are the strengths and elastic properties of the steel or stone as used, but in the case of cement it is only possible to say that if it is used under certain well defined conditions it may be expected that the strength and elastic properties of the resulting product will have certain values at different ages. The qualities of these other commodities are never spoken of as being thus and so at 7 or 28 days, but the strength of cement should never be spoken of unless coupled with a statement of its age since having been mixed with water. The interested public when it thinks of the quality of cement always associates the factor of time with quality and further bears in mind that quality exists in cement only potentially—to be developed in time through the agency of water.

Water is essential in making portland cement a valuable article of commerce. But what reactions take place when water is so added will of course depend upon what cement is. However, it is quite probable that these reactions are not common to cement alone but are of one or more of the types common to the action of water on any other solids which dissolve with difficulty. By analyzing cement after removal from water it is possible to demonstrate that water has entered into combination with it; by analyzing water removed at different intervals from contact with cement it can be shown that the latter is not only dissolving but is also decomposing —hydrolysis is taking place. By using coarse or fine ground cement, by carrying out the reaction at higher or lower temperatures, it is possible to accelerate or retard the water-cement reaction. Such are the usual phenomena which are taking place, and the fact that so little is known of the details is due to there being so little known about one of the reactants —portland cement, and what is known is so slightly disseminated.

It is not so well known as it should be that portland cement is not a definite compound. Neither is it a homogeneous mixture of several. It is however a heterogeneous mixture of at least three quite likely more or less impure compounds of lime with silica, alumina, iron and the lesser impurities of clay and limestone. The compounds predominating to the

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extent of 90 per cent or more are those of lime and silica in the molecular ratios of two or three of the former to one of the latter, and that composed of three of lime to one of alumina. The determination of the exact purity of these in finished cement is just as difficult as is the determination of the amount of each. Indeed it were better not to speak of determining either the purity or the quantity but rather to speak of estimating these. However it is possible to prepare the three major constituents in a pure condition or containing impurities not present in cement, study these when reacting with water and draw conclusions which are undoubtedly applicable to cement. This has been done. Also portland cements produced under close scrutiny have been made and these observed while reacting with water. Briefly all data show that depending upon the kind and amounts of certain constituents present in a cement, the quality of the

TABLE I—AMOUNT OF HARDENING OF SEVERAL CEMENTS AT DIFFERENT TIMES.

	Tensile Strength, lb. per sq. in.				Per Cent Combined H_2O in Neat Specimens				Chemical Analyses					
	24 hrs.	7 days	28 days	6 mos.	24 hrs.	7 days	28 days	6 mos.	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	B_2O_3
$2\,CaOSiO_2$—Neat	Soft	Soft	Soft	*538	Not determ'd			4.9	33.76	0.33	0.34	63.44	1.51	0.75
$3CaOSiO_2$ Neat	224	493	857	670	5.1	8.1	9.0	10.1	26.21	0.20	0.49	70.53	1.11	0.96
$3CaOSiO_2$ 1:3 Mortar	....	191	323	467	....	....	....	....	.....	.....	.....	.....	.....	.....
Cement A Neat	213	467	691	647	6.9	8.6	10.3	11.4	25.30	7.34	0.08	64.47	2.15	0.00
Cement A 1:3 Mortar	....	190	337	373	....	....	....	....	.....	.....	.....	.....	.....	.....
Cement B Neat	170	156	176	599	4.7	4.8	6.0	10.6	28.50	7.46	0.22	60.47	2.22	0.00
Cement B 1:3 Mortar	....	71	130	385	....	....	....	....	.....	.....	.....	.....	.....	.....
Cement C Neat	325	595	610	515	8.4	9.7	11.4	12.9	21.82	7.42	0.10	68.09	2.20	0.00
Cement C 1:3 Mortar	....	390	373	410	....	....	....	....	.....	.....	.....	.....	.....	.....

* Tests made at 1 year.

concrete made therefrom will be a function not only of the type and proportions of the aggregates used and the ratio of the cement to the water, but also of the time elapsed since the reaction started.

Technologic Paper 78 of the Bureau of Standards contains data which have been obtained in the study of certain of the constituents when made in a fairly pure form, as well as that obtained in the study of cement characterized by either high silica or high lime and very small amounts of impurities. They therefore contained either high or low amounts of one of the two silicates of lime. Certain of these data are reproduced in Table I. Note that the purer form of the dicalcium silicate was so soft at the end of 28 days after gaging with water that it was useless to try to test the specimens. However, the purer form of the tricalcium silicate approached values commensurate with those of portland cement at 7 and 28 days. Also note that adding amounts of alumina equal to those generally present in portland cement to the impurer forms of the low

limed silicate materially increased the hardening properties at any period over those of the dicalcium silicate. But still the low limed (high dicalcium silicate) cement has reacted with the water as shown both by the strength and the per cent of combined water at a decidedly slower rate than the high limed (high tricalcium silicate) cement. Particular attention should be directed to the fact that at the last period shown the strengths of all but the dicalcium silicate have approached equality to a surprising degree. But at the same time there should be noted the marked difference in the amounts of water that have been combined by the different

TABLE II—PROPERTIES OF 6 CEMENTS AT DIFFERENT AGES: LB. PER SQ. IN.

Cement No.			1	2	3	4	5	6
Tensile Strength, 1:3 Standard Sand		1 day	137	202	140	95	73	Soft
		3 days	202	283	242	208	148	48
		7 days	315	368	312	252	197	82
		28 days	425	416	368	350	332	205
		1 year	319	402	415	472	460	393
Compressive Strength, 1:3 Standard Sand, 2 x 4 Cylinder		1 day	360	510	345	255	55	40
		3 days	1455	1905	1240	760	480	100
		7 days	2695	2695	2085	1445	745	255
		28 days	3945	4060	3145	2895	2105	795
		1 year	5095	4565	5415	4010	4725	3595
Compressive Strength, 1:6 Crushed Stone Concrete, 6 x 12 Cylinder	7 days		1705	2015	1485	1240	465	95
	28 days	Dry closet	3075	2735	2655	2470	1635	620
		Air	2800	2785	2595	2245	1525	535
		Alkali	2565	2635	2275	1920	1340	455
	1 year	Dry closet	5210	4510	4885	5435	4775	3420
		Air	3355	2970	3165	3030	2790	1885
		Alkali	4080	3510	3970	4350	3760	2550
Per cent H_2O used in Concrete			7.65	7.78	7.75	7.75	8.82	8.23
Chemical Analysis of Clinker		SiO_2	21.60	22.33	23.18	23.98	25.02	26.11
		Al_2O_3	6.20	6.10	6.19	6.57	7.17	6.83
		Fe_2O_3	2.42	2.55	2.55	2.64	2.59	2.90
		CaO	67.50	66.09	65.29	64.16	62.91	61.57
		MgO	2.01	1.99	1.94	1.98	1.98	1.98
		SO^2	.06	.26	.10	.21	.46	.43
		Ig. Loss	.30	.55	.29	.13	.33	.30
Per cent Cement Passing No. 200			78.6	79.0	78.0	78.2	79.0	78.9
Petrographic Analysis	Per cent 2CaO SiO_2		9.0	13.0	32.0	65.0	77.0	88.0
	Per cent Remainder*		91.0	87.0	68.0	35.0	23.0	12.0

* Largely 3CaO SiO_2 and 3CaO Al_2O_3.

cements. Although this latter feature is not so much a part of this paper as the question of time, it does serve to show the difference in properties at different times of the constituents of cements.

No reference is made in this table to the deportment of tricalcium aluminate, the aluminate generally considered to be present in cement. This compound reacts as energetically with water as does quicklime and consequently does not permit of being made into strength specimens unless excessive amounts of water are used. Such procedure results in a specimen which has no more strength than slaked lime. But the effect of alumina in cements can be noted by reference to the table and comparing the purer compounds containing little more than traces of alumina with

the cements containing over 7 per cent. The marked increase in the rate of the reaction of dicalcium silicate and the rather slight increase in the rate of the tricalcium silicate is very evident.

This very brief abstract of the data of the technologic paper clearly shows that in the case of the purer compounds and pure special cements the strength and the reaction with water are decidedly dependent upon the time that has elapsed since the water was allowed to come into contact with dry cement constituents.

The data presented in Table II are abstracted from the results of some work at a commercial cement plant where the product was made under close scrutiny and the clinker carefully sampled, ground and made into test specimens. No data were obtained during the aging of the specimens regarding the amounts of water chemically combined but much more complete data regarding the rate of gain of strength was secured. A careful study was also made of the amounts of the several constituents present in each cement. These data are given in the table. While all constituents other than the dicalcium silicate are listed together, a study of the chemical analyses will show that the relative amounts of constituents other than the tricalcium silicate must have varied but slightly throughout, since the amounts of all oxides other than lime and silica indicated in the chemical analyses were about the same in all of the cements. Consequently in cement number 1 the major constituent in the "remainder" as well as in the cement was tricalcium silicate. However, as the silica increases and the lime decreases the amount of this constituent decreases until in cement 5 very little or none was present, the major constituent having become the dicalcium silicate.

As the same aggregates were used in making the specimens with all these cements produced in the same kiln from the identical raw materials (but used in different proportions) and ground to practically the same degree of fineness, the strengths developed by the concretes and mortars at any one time (age) are largely, if not entirely, dependent upon the relative amounts of the two constituents—dicalcium and tricalcium silicate. In the study of the data in Table I it was seen that each constituent reacted decidedly differently, especially at the early ages, the dicalcium silicate being particularly slow. In Table II this fact is again brought out. But it is also plainly shown that the rate of strength gain is far greater with the cements high in the low limed silicate than with the high limed one. Compare cement 2 with a dicalcium silicate content of 12.5 per cent with cement 5 with a dicalcium silicate content of 77.0 per cent, and note that the former gave in the concrete a strength of 2,015 lb. per sq. in. at 7 days and the latter but 465 lb. But on the other hand note that at the end of one year the former showed 4,510 lb. and the latter 4,775 lb. Thus one can be assured if a high early strength cement is desired it can hardly be obtained from a high silica cement. At the same time the choice of a high limed cement will give a high early strength cement, but one showing a slight rate of strength increase with time. The marked difference be-

tween cements as exhibited in concrete is accentuated at different times and to evaluate a cement it is necessary to consider the time at which the concrete itself will be evaluated—tested.

The mortar specimens indicated in Table II were stored in running water. The concrete specimens however were stored in three different manners—first, in the damp closet; second, in the damp closet for seven days and then on the roof of a building in Texas until tested; and third, in the damp closet for seven days and then in soil very rich in white alkali in New Mexico until tested. Precipitation in these localities is very slight and the humidity among the lowest in the United States. The data from these storage conditions are given more on account of the general lack of such data than their bearing on the present subject, although again even under such widely varying conditions of aging, time is a predominating feature of the quality of strength.

It is unfortunate that experimental methods in physical-chemistry have not been developed which are adaptable to the solution of the type of reactions which cement and water undergo, when they are mixed in the proportions used in actual practice. Only those methods have been applied which permit of study when the water is present in much greater amounts than the cement, whereas methods are needed to study the exactly opposite condition. A general idea of the extent to which cement and water will react under such conditions can be obtained from Technologic Paper 12 of the Bureau of Standards. But the problems under discussion in that paper are not of concern at the present time, excepting in so far as they show that the extent of the reaction is decidedly a question of the length of time of the reaction. In the greater part if not in all of such recent work the first observation has been made at the expiration of at least 24 hours. But at the bureau it was recently decided to start observations immediately on adding the water to the cement. The progress of the reaction was followed very largely by the determination of the pH value[1] at stated intervals. The more marked activity of the aluminates focussed attention on these and finally the ease of securing aluminate solutions from the commercial high alumina cements started some observations on these which are worth abstracting here, especially as it is possible to apply some speculative deductions from the data to portland cement.

It was early noted that on mixing one part of commercial high alumina cement with 20 parts of water not only were the amounts of

[1] The term pH may be briefly and not too exactly explained as follows: Water or aqueous solutions of electrolytes (solutions that conduct the current) are dissociated in part into hydrogen (acid) and hydroxyl (basic) ions and the product of the concentration of these at equilibrium is a constant 10^{-14}. The ordinary logarithm of the reciprocal of the hydrogen-ion concentration in an aqueous solution is denoted by the symbol pH. This for neutral water is equal to 7.0. The pH of acid solutions is less than 7.0 but since the product of the concentration of hydrogen and hydroxyl ions is 10^{-14}, it is permissible to construct a scale of acidity and alkalinity values of 0 to 14 and to speak of the hydrogen-ion concentration of an alkaline solution. The older conception of acidity or alkalinity more properly referred to the quantity of ionized and ionizable hydrogen or hydroxyl ions whereas the pH value refers to the ionized ions alone.

cement going into solution not following any apparent law, but also the pH values were apparently erratic. It was therefore decided to remove the water, at different intervals within the first 24 hours, from mixtures in which the same proportions of cement and water had been used. Furthermore the solutions were examined immediately on removal and after standing. The data thus obtained by Dr. L. S. Wells of the bureau's staff are produced graphically in Fig. 1.

This shows a number of unexpected and remarkable facts. There is a remarkably rapid solution of the cement in the water and this amount increases for about six hours. It should be noted that the ratio of lime

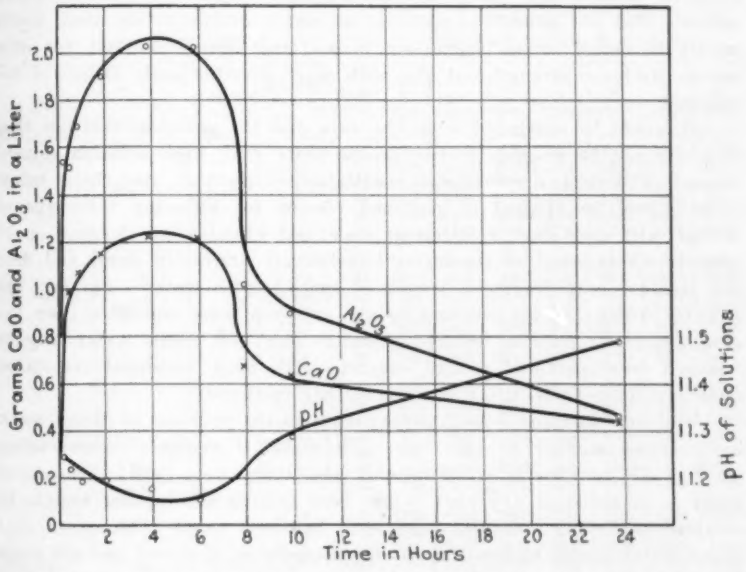

FIG. 1.—DATA OBTAINED FROM WELLS' TESTS ON CONCRETE MIXTURES.

to alumina in the solution during this time is very constant. Between six and eight hours, if the cement is allowed to stand in contact with the water, there is precipitated out of solution certain as yet unidentified crystalline aluminates of lime. Hence if the water is removed from a mixture of alumina cement and water which has stood somewhat longer than six hours, decidedly less solids are found to be present than in the water at six hours. For a short time after six hours this rate of precipitation proceeds very rapidly, then more slowly. But at the end of 24 hours contact of cement and water the latter will contain less solids than it did at the end of 15 min. contact. Note also that the pH value has decreased slightly from the 15 min. until the six-hour period, then raised very rapidly to eight hours and then increased to 24 hours.

Again time is shown as a most predominating factor in the reaction of alumina cements and water. The data would lead one to speculate as to what would be the effect if one removed the excess water from a sloppy concrete made from such cement between 15 min. and six hours after gaging. Unquestionably large amounts of dissolved cement would be removed which the data show would later precipitate out in the concrete if not removed. Is such removal advantageous or otherwise? No data in which high alumina concrete has been made under such conditions are at hand. Is it desirable to use such an excess water that so much cement can go into solution and later precipitate out? Here practice has apparently given the answer and has given much explicit evidence that high alumina cements will not permit of the use of water to the extent that would hardly be called excess in the use of portland cement without not only materially lower strength but also with possible rather early failure of the concrete.

It would be concluded from the data and the practical evidence that it would not be possible to use excess water with high alumina cement concrete and remove it without unsatisfactory results. But these deductions cannot be applied to portland cement for although this type of cement will also show solution of lime and alumina at the end of 15 minutes, the amount of the latter is relatively extremely small and does not increase with increased length of contact with water. Further, the silicates which are the bonding agents are even more insoluble than the aluminates. It may be deduced therefore that the excess water may be removed from portland cement concrete with most beneficial results at some certain periods before the setting has occurred.

Will any portland cement deport itself in the presence of excess water in the same manner at any time as it would if amounts corresponding more closely to what is called normal consistency were used? Can excess water be removed at any time before final setting and similar results be obtained as though a normal consistency had been used? This question of excess water cannot be gone into as intelligently as it should and not much data are at hand really covering it, but in Table III are presented data taken from Bulletin 5 of the Structural Materials Research Laboratory of Lewis Institute. Here there are presented results which were obtained in a study wherein for one of several fineness moduli using two kinds of aggregate the variables were time and water-cement ratio. Studying the data presented as a result of the work with the sand and crushed limestone aggregate, it is noted that the highest strength at seven days was obtained when a water-cement ratio of 0.81 was used; but at 28 days the highest strength resulted when the ratio was 0.90. While at seven days a water-cement ratio of 0.99 gave 31 per cent lower strength than did a ratio of 0.81, at six months these same ratios gave the same strength and at one year the wetter consistency gave a strength almost 20 per cent higher than the dryer one. Increasing the water-cement ratio from 0.63 to 1.35 gave a reduction in strength of a little more than 67 per cent at

seven days but at two or three years the reduction was approximately only 6 and 3 per cent, respectively. At five years the ratio of 1.13, which is getting very close to the water content that produces the undesired sloppy mixes, has given a strength of 4,220 lb. per sq. in.—a strength that would give any one assurance of permanency; but note that at seven days this ratio gave a strength of less than half the maximum produced by any of the water contents used. The concretes made from the pebble aggregates show the same character of relationships between ages and water-cement

TABLE III*—STRENGTH VS. CONSISTENCY: LB. PER SQ. IN.; 6 X 12-IN. CYLINDERS; 1:5 BY VOLUME

Aggregate			Mixing Water		Compressive Strength, lb. per sq. in.						
Kind	Size	Fineness Modulus	Relative Consistency	Water-Ratio	7 days	28 days	6 mos.	1 year	2 years	3 years	5 years
Sand-Pebbles	0 to 1½ in.	6.00	0.80	0.66	2230	3250	3870	4430	4860	5510	4540
			0.90	0.74	2020	3560	4240	4580	5230	6540	6310
			1.00	0.82	1660	3150	4370	4880	4960	5780	5870
			1.10	0.91	1260	2720	3770	4370	4490	5480	5530
			1.25	1.03	900	2220	3210	3330	3590	4370	4430
			1.50	1.24	520	1340	2080	2290	2750	3780	2790
	0 to ¾ in.	4.50	0.70	0.74	1060	1890	2460	2780	3110	3240	3230
			0.80	0.85	1070	2140	2820	3110	3590	3910	3610
			0.90	0.95	990	2020	2930	3120	3510	4050	4100
			1.00	1.06	790	1700	2730	2910	3450	3830	3850
			1.10	1.17	630	1530	2490	2530	3050	3380	3510
			1.25	1.33	470	1140	2030	2260	2790	2780	2910
			1.50	1.60	330	880	1510	1870	2050	2240	2070
	0 to No. 4	3.10	0.70	0.89	580	1060	1710	1830	2020	2190	2100
			0.80	1.02	530	1090	1740	1810	2350	2240	2240
			0.90	1.15	510	1120	1770	2030	2270	2320	2350
			1.00	1.28	420	990	1500	1940	2080	2260	2390
			1.10	1.40	350	810	1380	1520	1910	1850	2060
			1.25	1.60	260	580	1240	1470	1610	1840	1630
			1.50	1.92	180	490	910	1120	1170	1400	1320
Limestone	0 to 1½ in.	5.50	0.70	0.63	1280	1970	2490	2740	2580	3180	3540
			0.80	0.72	1350	2500	3350	3540	3760	3760	3730
			0.90	0.81	1590	2650	3820	3630	5260	5580	5060
			1.00	0.90	1360	2720	3480	4540	5110	5900	5740
			1.10	0.99	1100	2410	3820	4330	4860	5420	5610
			1.25	1.13	730	1610	3040	3290	3760	4220	4220
			1.50	1.35	420	1010	1710	2040	2400	3080	2560

* From Table IV, Bulletin 5, Structural Materials Research Laboratory, Lewis Institute.

ratio. Thus at seven days the pebble concrete with a fineness moduli of 6 and a water-cement ratio of 0.66 developed the maximum strength for all consistencies. At this time this ratio showed a strength 2.4 times that developed by the ratio of 1.03. But at five years the strength ratios for these two consistencies had fallen from 2.4 to somewhat less than 1.03. At this last period the maximum strength developed by any consistency was 1.4 times that developed by the consistency which at seven days gave maximum strength, although at seven days the strength ratio of the two was 0.90. Putting the relation of these two latter consistencies in another manner we have—the one giving maximum strength at seven days had

increased in strength 2 times in five years, while the wetter consistency giving the lower strength at seven days had increased 3.1 times its 7-day strength in five years. Still more strikingly the wettest consistency (a water-cement ratio of 1.24—equivalent to about 9.5 gal. of water per bag of cement) at five years had a strength about 5.4 times its 7-day strength. Such data would seem to prove quite conclusively that it is not possible to predict that because the consistency of the mix or the water-cement ratio used has produced at one age a maximum strength it will produce the maximum strength at all ages. On the other hand it would appear that time is a most effective obliterator of the effect of widely-varying consistencies.

Actual practice indicates that good concrete is being made daily with large excesses of water but the latter removed by overflowing from or leakage through the forms. For some data in respect to the latter, reference should be made to the *Proceedings* of this Institute, Vol. XVI (1920) containing the paper by Hull on "Fire Tests of Concrete Columns." In making certain of the columns (numbers 66, 67, 72, 89 and 90) the spiral reinforcement was covered with diamond mesh metal lath and the latter used as a form. Hull states, "with a consistency as wet as could reasonably be attained in hand mixing the quantity of *wet mortar* which escaped through the metal lath was not important." Also as shown in Table VII of Hull's paper, one of these columns when protected by two inches of a plaster of gypsum, lime and kiesselguhr withstood the four-hour fire test at the end of five months and when later tested cold gave a load of 5,345 lb. per sq. in. This would be considered most excellent strength for a 1:2:4 concrete column with 2 per cent vertical and 1 per cent spiral reinforcement, at the age of five months.

Such data would seem to show that the use of excess water produces results similar to that produced by amounts corresponding to normal consistency provided the excess water is removed within a few hours after the initial mixing of the dry materials and the water. They do not throw any light on what takes place if the excess is retained in the mix for some time after, when the stiffening of the mass should occur. Under such conditions as the latter, has the excess water simply dispersed the cement grains to a degree that they are not in sufficiently close contact to permit of the cementitious products formed by the reaction to bond the inactive particles of the cement or aggregate together or has the reaction proceeded to a degree or in a manner beyond or different from that which follows in the presence of the normal amounts of water producing the desired bonding properties?

The swelling which takes place when cement is shaken with large excesses of water is generally known. If this water is removed from this swollen mass by filtration or evaporation and the residue dried, a soft pulverent mass remains. However this swollen cement is used in making such products as asbestos boards and shingles, by mixing it with asbestos, filtering, sheeting and compressing. The production of this firm hard

coherent sheet would seem again to indicate that excess water thoroughly removed at the proper time does not produce any deleterious effects. Apparently until after the elapse of a certain time the amount of water used is a matter of choice and convenience. The example cited of the asbestos shingle is particularly apropos in view of the fact that two directly opposite methods of manufacture are used. In the one procedure

TABLE IV—EFFECT OF REMOVING EXCESS WATER ON STRENGTH: 1:2:4 PEBBLE CONCRETE

Time of Molding	Kind of Molds	Consistency			Flow	Slump	Compressive Strength, lb. per sq. in.	
			Per Cent H_2O	H_2O/Cement			7 days	28 days
Immediately after mixing	Steel	Dry	8.0	0.90	24	0.4	1810	3560
							1870	3410
							1850	3660
							Av 1840	3540
	Steel	Wet	11.6	1.31	150	8.3	500	1120
							465	1100
							440	1080
							Av. 470	1100
	Plaster	Wet	11.6	1.31	150	8.3	1340	1750
							1460	1990
							1490	1940
							Av. 1430	1890
3 hours after mixing	Plaster	Wet	11.6	1.31	102	7.4	1790	2750
							1890	2460
							1860	2890
							Av. 1850	2700
	Steel	Wet	11.6	1.31	102	7.4	625	1390
							650	1270
							570	1190
							Av. 615	1280

Sieve Analysis of Sand		Sieve Analysis of Pebbles	
Sieve No.	Per Cent Retained	Sieve No.	Per Cent Retained
4	2	1 in.	0
8	18	½ in.	30
16	32	¼ in.	75
30	50	4	100
50	84		
100	95		

outlined above of filtering and compressing, a very wet mixture is used, in the other a very dry mixture is compressed. The results are products of almost identical physical properties.

Recently a short series of tests of a preliminary nature were made at the bureau covering both of these points—the use of excess water immediately removed after mixing and excess water removed three hours after

mixing. Such concrete was compared with similar concrete made at the same time but the water all retained in the mix during hardening and with similar dry mixed concrete. The excess water was removed by casting certain of the specimens in cylindrical plaster-of-paris molds. The remainder of the specimens were cast in steel molds in which all joints were carefully sealed to prevent all seepage of water. All specimens were kept in the molds for three days, then in the damp closet for four days, and then in the air of the laboratory until tested. All data are given in Table IV. Little comment is needed. But please note that the dry mix (water-cement ratio 0.90) gave no different strength at 7 days than did the wet mix (water-cement ratio 1.31) when the excess water was removed by casting the concrete three hours after gaging, all water having been prevented from evaporation during the three hours by keeping the concrete in tightly-closed cans. When, however, the wet mix was immediately cast in the absorptive molds the strength at 7 days was about 78 per cent of the dry mix or the mix in which the excess water had been allowed to react for three hours. The wet mix cast in the non-absorptive molds gave much lower strengths. At 28 days the dry cast specimens were decidedly the stronger, but the specimens cast three hours after gaging gave strengths far in excess of that generally credited to such a water-cement ratio.

This short series of tests in connection with those from Bulletin 5 of the Lewis Institute rather strikingly answers in the affirmative the two questions propounded above. Excess water can be used and does at certain times (ages) develop properties corresponding very closely to those developed by normal consistencies. And excess water can be removed from mixes at certain times before final set and results obtained at certain ages equivalent to those resulting when normal consistencies are used.

This paper may seem to be unduly adversely critical of certain extensively accepted generalizations which have come to be considered almost laws. But it is not intended to do otherwise than call attention to the fact that the making of concrete is no other than starting on their way certain chemical reactions taking place between a number of solids (cement) and water. These are typical of such reactions in general, and in such nothing is more important than the rate at which they proceed towards completion; and rate cannot be defined without the concept of time. Concentration, which in this special case is the ratio of reacting solids (cement) to solvent (water) and is therefore the water-cement ratio, is a function of time and is not a constant over any great period of time, starting from the instant water is added to the cement until the time that equilibrium may be reached in an atmosphere of constant humidity. Hardening or the acquiring of the several properties desired of concrete is solely a question of the ratio of bonding agent (partially acted upon cement) to the amount of water either liquid or gaseous present, and both of these variables are subject to change in quantity as time progresses. In all respects therefore time is a most important factor in the study and use of cement.

Time as a Factor in Making Concrete—Discussion.

F. R. McMillan* (*By Letter*).—Mr. Bates has referred to some of the data included in Bulletin 5 of the Structural Materials Research Laboratory.† These data have been quoted to show that the time element is not of the same importance for concrete of different consistencies. From the statements which he has made it would appear that concretes of the wetter consistencies gain in strength more rapidly than those of drier consistencies, and even equal the drier in strength at the later ages. If this were true, it would in a certain measure tend to refute the theory that the quantity of mixing water is a primary factor in the quality of concrete. The following quotation from the last paragraph of his paper would seem to indicate that Mr. Bates himself was of the opinion that this theory does not hold except perhaps for the earlier ages: Mr. McMillan.

> "This paper may seem to many unduly adversely critical of certain extensively accepted generalizations which have come to be considered almost laws, but it is not intended to do otherwise than call attention to the fact that the making of concrete is no other than starting on their way certain chemical reactions taking place between a number of solids (cement) and water."

A careful study of the data in Table III of Mr. Bates' paper, taken from Table IV, Bulletin 5, does not warrant the conclusions which he has drawn, nor does it destroy confidence in the water-cement ratio law of concrete quality. The water-cement ratio law of strength developed by Abrams can best be stated in the following words:

> "For given materials and conditions of manipulation, the strength of concrete is determined solely by the ratio of the volume of mixing water to the volume of cement so long as the mixture is plastic and workable."

While this law is usually given in reference to 28-day strengths, all of our tests show that for workable mixtures, strengths at ages greater than 28 days exhibit the same general agreement within themselves and conform to curves very similar and nearly parallel to that representing the relation at 28 days. In all of Abrams' writings, emphasis is placed upon the importance of the item of workability if the strengths are to be compared on the basis of this law.

The error into which Mr. Bates has fallen in the use of the data from Bulletin 5 is in the comparison of unworkable mixtures of low water-ratio with workable mixtures of high water-ratio. This error is easily made and would not be detected by a casual reader. It is only when all the

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* Director of Research, Portland Cement Association, Chicago, Ill.

† Modulus of Elasticity of Concrete, by Stanton Walker, formerly Associate Engineer of the Structural Materials Research Laboratory.

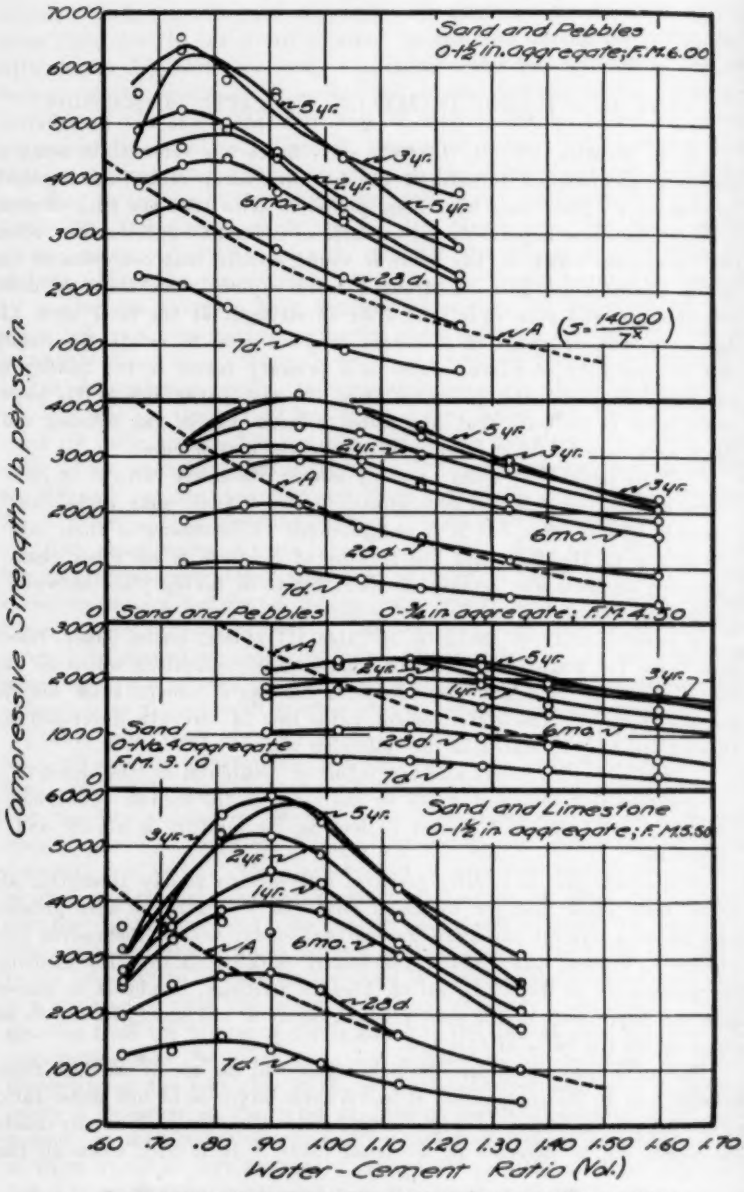

FIG. 1.—WATER-RATIO STRENGTH CURVES OF VARIOUS MIXTURES.

data in his Table III are plotted that this error is revealed and the significance of the different elements is apparent. In Fig. 1 are plotted the water-ratio strength curves for all the mixes in this table.

Two features should be observed in the study of Fig. 1. In each group of curves representing the different aggregate combinations for the various water-cement ratios and ages, it will be noted that in every instance the strength begins to increase with increase in water-ratio until a high point is reached beyond which the strength falls off with increased water content. For example, in the lower group of curves representing the 0—1½-in. limestone aggregate (F. M. 5.50), it will be noted that the maximum strength for all ages occurs at water-ratio of about 0.90. The mix for this 0.90 water-ratio is indicated in the table as of relative consistency 1.00; this corresponds to a slump of about ½ to 1 in. and, as will readily be recognized, is not a particularly workable mixture. This is especially true for crushed rock coarse aggregate in a 1: 5 mix graded to a fineness modulus of 5.50. For the mixes drier than this the conditions are very much worse. For this group of data, the true workability begins at about relative consistency 1.10 (water-ratio 0.99). For higher water contents the mixes are workable and as will be seen from the curves they show the characteristics of the typical 28-day water-ratio strength curve.

In the upper group of curves Fig. 1 representing the pebble coarse aggregate, 0—1½ in., F. M. 6.00, the high point of the group falls at a water-ratio of between 0.75 and 0.80. In this, as in the lower group, the curves beyond the high point are in quite fair agreement as to form and relative position.

In the second group of curves in Fig. 1, 0—¾-in. aggregate, F. M. 4.50, the same characteristics will be observed, and the curves are somewhat more regular. As in the upper group, fewer of the points are to the left of the high point. This is because of the better workability of the mixtures due to the lower fineness modulus.

In the third group, that for the fineness modulus 3.10, the values are all somewhat low. This is a particularly good demonstration of the effect of non-workability on the strength of concrete. These are mortar mixes and as will be seen from the notation at the head of Table III, are very lean (1: 5). These mixes do not puddle in the forms until the water-ratio is quite high. For the consistencies around the 1-in. slump, holes are still visible when the tamping rod is removed, showing that the mass does not consolidate readily.

In Fig. 2, the tabular values from Table III have been plotted to show the relation of strength to age (to logarithmic scale). In plotting this figure, all of the non-workable mixes as revealed from Fig. 1 have been omitted. The mixes excluded are those falling to the left of the high point of the curves, and the 1: 5 mortar mixes from 0-4 sand of F. M. 3.10. It will be seen from the groups of curves in Fig. 2 that the relation between strength and age is surprisingly regular and that in all cases the mixes of lower water-ratios are above those for higher water-ratios. This is

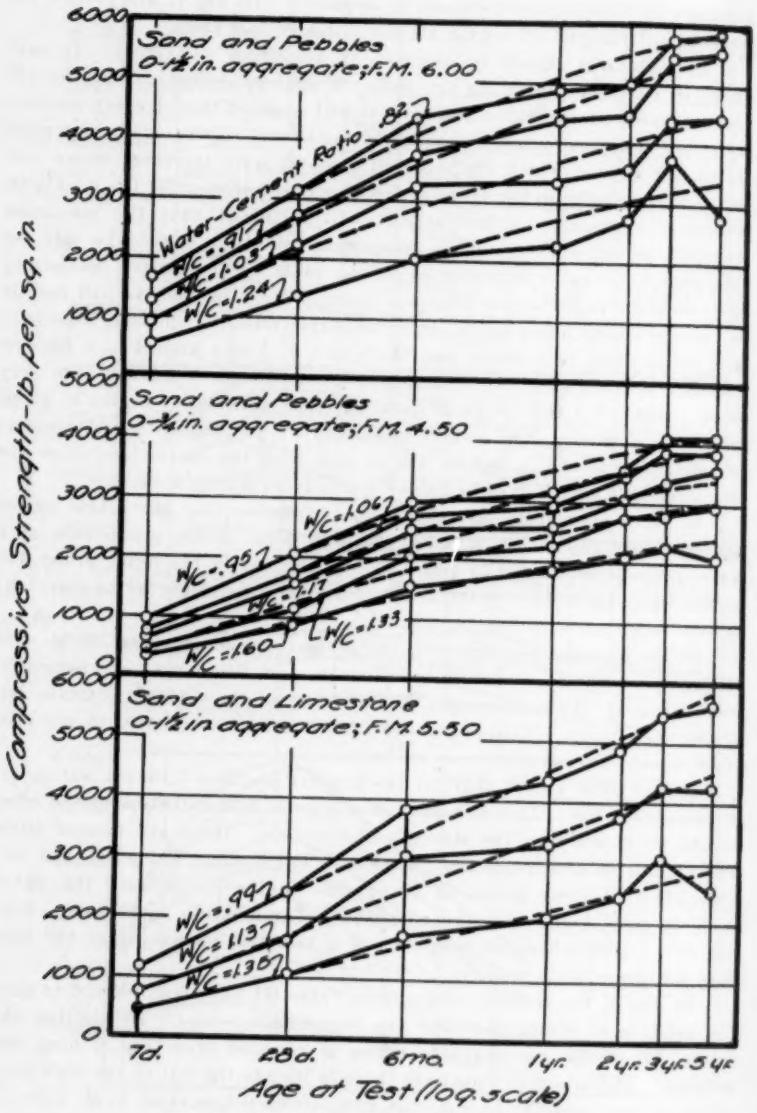

FIG. 2.—RELATION OF STRENGTH TO AGE.

directly opposite to the conclusions Mr. Bates has drawn and shows the basic character of the water-ratio law. Furthermore, it is significant that the lines for the different water-ratios are diverging, thus showing that the margin of spread between the higher and lower strength concretes continues to widen as time goes on. This is typical of many other tests from this laboratory showing the continued increase in strength where the specimens are cured moist.

It is interesting to study Fig. 2 following the procedure often used in comparing concrete strengths at different ages, that is, by percentage increase. In the upper group of curves, if we compare the mixes of water-ratios 1.03 and 0.82, we find the water mix has a strength at 7 days of 53 per cent of that shown by the drier mix while at 5 yr. its strength is 75 per cent of the drier. This is often interpreted as indicating that the wetter mix is approaching the drier in strength as time goes on. Plainly, this is not the case since the lines are diverging, showing a wider and wider spread in pound per square inch in favor of the drier mixture.

A similar comparison for 1.60 and 0.95 water-ratios in the second group of curves shows percentages of 30 and 53, respectively, for 7 days and 3 yr. Also, for the lower group, the comparison of the wettest with the driest mix shows 42 per cent at 7 days and 50 per cent at 2 yr. In both of these groups also the lines are diverging, indicating that the wetter mixes, though gaining in strength, are not holding their margin with the drier mixes.

That these concretes behave in the manner shown in Fig. 2 and not in the hit or miss fashion indicated by the few isolated comparisons which Mr. Bates has made, does not controvert Mr. Bates' central theme that time is a vital factor in the development of concrete strength. In fact, it seems to the writer that it is a most reassuring confirmation of the importance of the time element—reassuring not only because it is consistent with all the other known facts of concrete mixtures, but also because it removes that element of accidental relationships which would seem to govern if the facts were as indicated by the few examples which Mr. Bates used.

Mr. Bates has made the point from the tests reported in his Table IV that leakage of water from the forms or removed in other ways from the mix during the early period, influences the strength. This is exactly in accord with our conception of the water-cement-ratio—strength law. Any water that is removed from the mass while it is still plastic so that as the water is extracted the mass can consolidate, is no longer effective in the mix. Thus, it is the water-cement ratio as the concrete lies in the forms in a period shortly after the placing that determines the strength, and not necessarily that introduced at the mixer.

Water-Cement Ratio as a Basis of Concrete Quality.

By Duff A. Abrams.*

Water-Ratio—Strength Relation for Concrete.—Tests carried out by the writer (later confirmed by many other investigators) have shown the controlling influence of the quantity of mixing water on the strength and other properties of concrete, regardless of mix, size or grading of aggregates, consistency of concrete, etc., so long as the concrete is plastic.[1] It is assumed that a satisfactory cement is used and that aggregates which are clean and of suitable strength and durability are available. The quantity of mixing water may be expressed in terms of the water-ratio; that is, the ratio of the volume of water to the volume of portland cement. The actual amount of water to be used in each batch may be measured in U. S. gallons per 94-lb. sack of cement.

An excess of mixing water results in concrete of low strength and inferior resistance to weather and other destructive agents. Increasing the quantity of mixing water beyond that necessary to produce a workable mix dilutes the paste and produces a dispersion of the cement particles so that adhesion between cement particles and between cement and aggregate is greatly weakened. The use of more cement accomplishes no useful purpose, unless the water-ratio is thereby reduced.

Where the water-ratio is controlled by accurate measurement of quantities of materials and correction is made for water carried by aggregate, the compressive strength of plastic concrete may be represented by a curve of the form

$$S = \frac{A}{B^x}$$

where S is the compressive strength of the concrete,

x (an exponent) is the water-ratio,

A and B are constants for given materials and test conditions.

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[1] The laws on which the water-ratio method of designing concrete mixtures is based were developed in the experimental studies of concrete carried out at Lewis Institute during the decade 1916-1926 through the co-operation of the Portland Cement Association and the Institute. This co-operative arrangement was terminated on June 30, 1926; however, the researches are being continued in the Research Laboratories of the Portland Cement Association, Chicago, and the International Cement Corporation, New York.

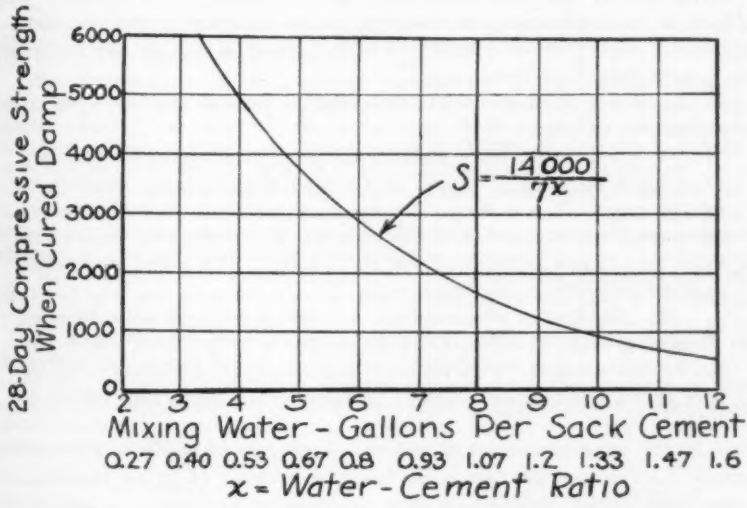
* Director of Research, International Cement Corp., New York; Former Director, Research Laboratory, Portland Cement Association, Chicago.

Using an average portland cement concrete cured in a damp condition at normal temperature and tested at 28 days, the relation between compressive strength and water-ratio is approximately

$$S = \frac{14000}{7^x}$$

This curve is shown on the accompanying diagram.

Other Properties of Concrete.—While compressive strength is not the only important requirement of concrete, it has been shown that flexural strength, resistance to wear, as well as many other desirable properties, are influenced by variations in water-ratio in much the same way as the



compressive strength. There is also a close co-ordination between the 28-day compressive strength and the resistance of concrete to sulphate soils and waters as shown by field tests in Colorado and South Dakota. An important property of concrete is its resistance to weathering which is measured to a large extent by its permeability. Prof. M. O. Withey of the University of Wisconsin has shown that permeability decreases with increased compressive strength and that maximum watertightness is obtained over a narrow range of water-ratio.

Water-Ratio Method of Designing Concrete Mixtures.—The water-ratio-strength relation furnishes a practicable method of securing concrete of a predetermined quality after suitable materials have been selected.

This method consists of two essential steps:

(1) Fix the water-ratio which will produce the required strength (free water held by the aggregates must be included as mixing water).

(2) Determine by trial or calculation the amount of fine and coarse aggregates which can be most economically used with a fixed water-ratio to produce concrete of the required workability.

It is assumed, of course, that the concrete will be properly mixed, placed, and cured.

It should be emphasized that this method fully recognizes the importance of *workability*. In making concrete, we must *pay for workability* in exactly the same way as we pay for strength or other properties. Workability may be measured by the "slump" or "flow" test.

Defects of Older Form of Specifications.—The older concrete specifications based on fixed proportions of cement, fine and coarse aggregates and rigid limitations on size and grading of aggregate frequently resulted in inferior and at the same time more costly concrete. The difficulty was due to a misapprehension of the true nature of concrete and resulted in the entire neglect of the quantity of mixing water, which we now recognize as a most important factor.

The older type of specifications tended to increase the cost of the concrete for two reasons:

(1) In general, more cement was used than would be necessary for a given strength, if proper limits were placed on the quantity of mixing water. It was formerly considered axiomatic that the quality of concrete was a function of the *quantity of cement*, whereas we now know that it is a function of the water-ratio. Rich mixes give better concrete only due to the lower ratio.

(2) Aggregates were required to conform to rigid limitations on size and grading in order that some arbitrary proportions might give the desired strength. As a result, it was frequently necessary to transport aggregates long distances or to waste certain sizes, thus adding to the cost.

By the water-ratio method failure of aggregate to meet the old requirements for size and grading is no longer considered of prime importance, since usual variations can be largely compensated by changes in mix, without sacrificing the strength or quality of the concrete. Almost any two fine and coarse aggregates may be combined in such a manner as to give good results.

New Type of Specifications for Concrete.—The practicability of the method was shown by its use as the basis of a specification covering the construction of the reinforced-concrete building of the Portland Cement Association, Chicago, during the winter 1925-26. The quality of concrete was fixed by three simple requirements.

(1) *Mixing water* limited to 6 or 7½ U. S. gal. per sack of cement for concrete of 2,900 or 2,000 lb. per sq. in. respectively at 28 days.

(2) *Proportions* of cement to fine and coarse aggregates determined solely by the requirements of workability.

(3) *Aggregates*—quantity of coarse not less than fine and not more than twice fine.

This method permits the widest latitude in quantity and size and grading of aggregates.

A report on the tests made on this building was given in a paper by McMillan and Walker on the "Use of the Water-Ratio Specification on the Portland Cement Association Building" (*Proceedings*, American Concrete Institute, 1926).

TYPICAL SPECIFICATION FOR CONCRETE.

Quantity of Mixing Water.—Concrete for structural members shall be proportioned to give the necessary workability without exceeding the following quantities of mixing water:

28-day Strength of Concrete Shown on Plans	Maximum Quantity of Mixing Water per 94-lb. Sack of Cement
2,900 lb. per sq. in.	6.0 U. S. gal.
2,000 " " " "	7.5 " "

(Where more severe exposure to weather must be met, or unusually high stresses are to be used, concretes of other classes may be specified by use of the curve shown on the attached diagram.)

The water-ratios are the maximum permissible. The mixes shall be proportioned for somewhat lower ratios so that with the normal fluctuations from batch to batch, these ratios will not be exceeded. Moisture held by the aggregate must be included in computing the water-ratios. Water absorbed by the aggregate in a period of 30 min. may be deducted in computing the water-ratio.

The water-ratios specified shall not be changed except by the engineer. In the event that it is necessary to change the water-ratios from those specified, adjustment covering amount of cement and aggregates affected will be made as an extra or a credit under the provisions of the contract.

Measuring Moisture in the Aggregate.—Moisture in the aggregate shall be measured by a method satisfactory to the engineer, which will give values within 1 lb. for each 100 lb. of aggregate.

Concrete Proportions and Consistency.—The proportions of aggregate to cement for concrete of the water-ratios specified shall be such as to produce concrete that can be puddled readily into the corners and angles of the form and around the reinforcement without excessive spading and without segregation or undue accumulation of water or laitance on the surface. In no case shall concrete be placed which shows a slump outside of the following limits:

Part of Structure	Slump—Inches	
	Minimum	Maximum
Caissons	1	4
Heavy Walls, Slabs, Beams	3	7
Thin Walls and Columns	4	8
Pavements	1	3

The proportion of fine and coarse aggregate shall be such that the ratio of the coarse to the fine shall not be less than 1 nor more than 2[1], nor shall the amount of coarse material be such as to produce harshness in placing or honeycombing in the structure. When forms are removed, the surface and corners of the members shall be smooth and sound throughout.

[1] The following limits in the proportion of coarse to fine aggregate are suggested for general use:

Maximum Size of Coarse Aggregate—Inches	Ratio of Coarse to Fine Aggregate on Basis of Dry and Rodded Volumes	
	Minimum	Maximum
⅜	0.4	0.8
¾	0.6	1.5
1 and over	1.0	2.0

Control of Proportioning.—The methods of measuring materials shall be such that the proportion of water to cement can be closely controlled during the progress of the work and easily checked at any time by the engineer or his representative.

To avoid unnecessary or haphazard changes in consistency, the aggregate shall be obtained from a source which will insure uniform quality and grading during any single day's operation, and they shall be delivered to the work and handled in such a manner that variations in moisture content will not interfere with the production of concrete of a reasonable degree of uniformity.

Tests of Concrete.—Frequent tests may be made by the engineer throughout the work to determine the quality of concrete. These tests will be made at the expense of the owner, and will, in general, be made on 6 x 12-in. concrete cylinders loaded in compression at 7 and 28 days, in accordance with the Standard Method of Making and Storing Specimens of Concrete in the Field (Serial Designation C31-21) of the American Society for Testing Materials.

The contractor shall co-operate in every way to the end that concrete of the desired quality be obtained. He shall provide, at cost, such housing as may be required for testing equipment and storage of test specimens; such cost to include only labor and materials actually used.

Portland Cement shall conform to the Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials (Serial Designation C9-26).

Fine Aggregate shall consist of natural sand having clean, hard, strong, durable, uncoated grains, free from injurious amounts of dust, lumps, soft or flaky particles, shale, alkali, organic matter, loam, or other deleterious substances. The sand shall be of such sizes that it shall all pass a ⅜-in. sieve, at least 15 per cent shall be retained on the No. 8 sieve, and the fineness modulus shall not exceed 3.20.

Coarse Aggregate shall consist of gravel, crushed stone, crushed air-cooled blast-furnace slag weighing not less than 70 lb. per cu. ft. or other approved inert materials, having clean, hard, strong, durable, uncoated particles, free from injurious amounts of soft, friable, thin, elongated or laminated pieces, alkali, organic or other deleterious matter. Coarse aggregate shall not have more than 10 per cent finer than the No. 4 sieve and the maximum size shall not exceed —— in.[1]

Storage of Aggregates.—Aggregates shall be stored so as to avoid inclusion of foreign materials. Frozen aggregate or aggregate containing lumps of frozen materials shall be thawed before using.

Water for concrete shall be from the city water supply or other source approved by the engineer.

Mixing Concrete.—The concrete shall be thoroughly mixed in a batch mixer of approved type. The mixer shall be equipped with suitable charging hopper. A water-measuring device shall be provided. The mixing of each batch shall be provided. The mixing of each batch shall continue for at least one minute after all the materials are in the mixer during which time the mixer shall rotate at a peripheral speed of approximately 200 ft. per minute.

Depositing Concrete.—Concrete shall be handled from the mixer to place as rapidly as practicable and in a manner that will prevent segregation of the ingredients. It shall be deposited in the forms as nearly as practicable in its final position to avoid rehandling. Concrete as it is deposited shall be puddled with suitable tools until forms are completely filled and reinforcement is thoroughly embedded in the mass.

Concrete shall be deposited continuously and as rapidly as practicable until the unit of operation, approved by the engineer, is completed.

Concrete when deposited shall have a temperature of not less than 40 deg. F. and not more than 120 deg. F. In freezing weather suitable means shall be

[1] This value must be supplied by the engineer.

provided for maintaining the concrete at a temperature of at least 50 deg. F. for 72 hours or more after placing, or until the concrete has thoroughly hardened. The methods of heating the materials and protecting the concrete shall be approved by the engineer. Salt or other chemicals or other foreign materials, shall not be mixed with the concrete for the purpose of preventing freezing.

Protection of Concrete.—Exposed surfaces of concrete shall be protected from drying for a period of at least 7 days after being deposited.

CONCRETE OF HIGH EARLY STRENGTH.

The water-ratio method of designing concrete mixes has shown the possibility of producing concrete of high strength at early ages. In many instances it is desirable to place concrete in service 2 or 3 days after mixing. The important factors which tend to produce high early strength are:

(1) Use of as low water-ratio as practicable.

(2) Increasing the quantity of cement, which in turn, permits use of still lower water-ratio.

(3) Use of aggregate graded as coarse as practicable.

If the quantity of mixing water is reduced to 5 or 4½ gal. per sack of cement and the quantity of cement is increased to, say, 2 bbl. per cu. yd., concrete can be produced which will, at normal temperature, give a 3-day strength of 2,500 lb. per sq. in. or more. Such concrete will generally require light tamping for proper placing and finishing.

In a number of instances concrete pavements constructed by this method have been opened to traffic on the third day after placing.

CONCLUDING REMARKS.

All of the properties and behavior of concrete may be explained by two factors:

(1) The "cementing value" of the cement used.

(2) The dispersion of the cement particles when concrete reaches its final place.

Any manipulation or device which will enable one to place concrete with a lower water-ratio or to eliminate the excess water after the concrete has been placed is effective in increasing the strength by reducing the water-ratio at the time of setting of the concrete.

The following methods have been found effective:

(1) Use of stiffer mixes which are placed by:

- (a) Tamping,
- (b) Pressure,
- (c) Vibration,
- (d) Impact (as in cement gun).

(2) Use of plastic mixes from which excess water is removed before cement sets by:

- (a) Absorbent forms,
- (b) Absorbent coverings or added dry materials,
- (c) Roller (as for concrete roads),
- (d) Centrifugal force (as in spun pipes and poles).

Time as a Factor in Making Concrete Pavement.

By T. H. Johnson.*

I have been asked to relate my experience in the use of the time theory coupled with the water-cement ratio, and how it has enabled me to over wet my concrete as the best means of placing it, and subsequently to remove the excess water as the best way of establishing the correct water-cement ratio in the concrete at the time of final set.

In making such an attempt I am sensible to the fact that I am calling your attention to a phase of concrete pavement construction heretofore entirely neglected, and suggesting practices that may seem to you not only unorthodox, but illogical and impractical as well. It is assumed that mine will be the common experience of those who represent a departure from the established order.

We began laying concrete pavement at Sioux City in 1911, three years before the tentative standard paving specifications were formulated, and our specifications, such as they were, were one of the ten sets used as a basis for that work.

In 1911 there was nothing to be found that might be used even as a foundation for a set of paving specifications, and we were compelled to begin at the beginning, and write our own. Experience has an established reputation as a teacher, and we began to learn from her. By the time the proposed standard specifications were available, in 1914, we had laid the foundation for a method of our own as well as thirty miles of pavement, and we have since continued to be guided by our own experiences, even though they have led us off at a tangent from the well-beaten paths of common practice.

Now, after 16 years' experience doing things in a way that is different, we are laying a pavement, embodying in its construction, it is confidently believed, a fuller compliance with the proven facts regarding the nature and behavior of concrete, a larger appreciation of present day knowledge of the principles involved, and of the multitudinous conditions that surround and affect the problem of pavement construction, than does the pavement being laid under other methods or practices.

One of the chief differences of our method is that it stresses that part of the paving process that follows the placing of the concrete, while the common practice stresses that portion that precedes it. Thus we introduce what seems now to have been denominated the "time factor," and clearly

* Sioux City, Iowa.

distinguish between the making of the *concrete*, and the making of the *pavement*.

In 1914, the editor of *Cement Era* visited Sioux City. Later, in an article under the title "Sioux City Pavement an Anomaly," he said: "After returning to Chicago, I took occasion to talk to several people who were familiar with the situation at Sioux City. They all seemed to be of one mind on two points: First, that there was no possibility of a dispute on the good quality of the Sioux City concrete pavements, and second, that in view of the disparity between the Sioux City practice and the ordinarily accepted standards, it was impossible to satisfactorily explain this excellence."

In 1916, one of the editors of *Engineering News* spent a couple of days in Sioux City, studying the concrete pavements and the method of laying them, and embodied his findings and his conclusions in a five-page article appearing in their issue of July 27, 1916, and a page editorial in the issue of Aug. 3, following, under the caption, "A Revolutionary Improvement in Making Concrete Roads." Among other things, he said, "It is no exaggeration to say that the concrete roads and streets of Sioux City represent an important advance over anything heretofore attained in concrete road construction within our knowledge."

Since 1916, ten of the most fruitful years in concrete experience have passed. More than 2,000,000 additional yards of this pavement have been laid, each year noting an improvement in quality due to the added experience, so that the pavement being laid now is the best of its kind that has ever been laid and the expectation is that a better pavement will be laid next year than was laid last year. The present method of laying it is the antithesis of the common practice in laying concrete pavements.

This pavement is laid on a dry, lightly rolled, absorptive base; common practice requiring a thoroughly rolled, saturated, non-absorptive base. This pavement is a lean mix, 1 : 2½ : 4½, with coarse aggregate up to 3 in., of a wetter consistency; common practice requires a rich mix, 1 : 1½ : 3 to 1 : 2 : 4, with coarse aggregate up to two or rarely 2½ in., and a drier consistency that requires tamping. This pavement is finished by applying a rich, dry mixture of cement and sand that withdraws the excess water within the first few minutes after placing, so that the workmen finishing it stand directly on the pavement without sinking into the concrete the depth of their shoe soles, and continue working it for an hour or more; the common practice requires haste in the finishing operations, using bridges for the finishers, and when completed, in 10 to 15 min., the concrete is still soft and mushy.

This pavement when finished has the correct water-cement ratio. It is dense because simultaneously with the removal of the excess water, and while the mortar portion still is in its most plastic state, the slab is thoroughly manipulated and massaged with heavy floats and the belt, reaching to the very bottom and thoroughly annealing the entire slab. This treatment, occupying the interval between placing and the final

manipulation, in connection with the application of a rich dry mixture, removes all excess moisture and produces a surface that is hard, dense, tough, non-abrasive and impervious.

These results are made possible by the introduction of the time factor, which is just as necessary in laying concrete pavement as it is in making bread.

With crushed rock at $2.50 per ton, sand $1.50 per ton, cement $2.40 per bbl., machine men at $50 per week, cement finishers $1 per hour and common labor at 40¢ to 50¢ per hour, this pavement can be laid 6 in. thick and with a five-year guarantee for less than $2 per square yard, and will test to average around 5,000 lb. per sq. in. It was laid in Sioux City last year under the above conditions as low as $1.69.

There is now in use of this pavement the equivalent of 300 miles of 18-ft. highway pavement, from one to fifteen years old, all laid under a four or five-year guarantee, and so far as this writer knows no contractor has ever been required to do any maintenance work during or at the end of the guarantee period. So far as he knows, there has never been a heaved joint nor a "blow up" in this entire area. Perhaps two-thirds of this area has been laid on loess soil, but there are no such cracks to be found in it as are described in *Public Roads* of August 1925, in an article regarding the difficulties of laying highway pavement on loess soil in northwestern Iowa. At rare intervals on very limited areas a few fine checks 1 to 6 in. long and seldom more than an eighth of an inch deep have developed. But these usually close as the core of the slab dries out. If they do not they are easily cured permanently.

In 1920, a little city in an adjoining state put in eight miles of this pavement, and as some of the streets to be paved were portions of the state highway system, the highway department was asked to approve the specifications. It declined to do so on the ground that these specifications were so at variance with the standards it was requiring of other towns, it would not be fair to them. The town council and a number of leading citizens had visited Sioux City to see the pavement there, and on a showing that this pavement could be laid for them at a saving of 75¢ per yard, or a total of $120,000, as compared with state specifications, they decided to use it.

When the pavement had been down and in use four years, the highway department tested it, among other things removing and testing 27 cores. The pavement was 6 in. thick and the cores 4½ in. in diameter. To make them comparable with the usual test cylinder having a length twice the diameter, 50 per cent was added to their length when capping the ends with 1:4 cement mortar, for testing. The coarsest aggregate used in the pavement was 3 in., so that the diameter of the cores was only one and one-half times the size of the coarse aggregate, instead of three times, to yield a fair test.

Notwithstanding these unfavorable conditions, and the further fact that the mixture was 1: 2½: 4½, these cores developed an average crush-

ing strength of 5,065 lb. per sq. in. Eliminating the lowest two tests, which ran about 40 per cent below the average, two-thirds of the remaining cores tested within 10 per cent of the average. Even though this pavement was only 6 in. thick, entirely without steel reinforcement, and laid with a mixture containing 25 per cent less cement than required by the state specifications, the specifications were approved and the town refunded $45,000 for the 18-ft. strip of highway pavement.

The contractors laid this pavement under a five-year guarantee, and at the expiration of this time they and their guarantee bond were released without being asked to spend one dollar for maintenance. The experiences of this town are typical of those of other towns, both larger and smaller, using this pavement.

During the past few weeks I have read with more or less care the published *Proceedings* of the Institute back to and including 1917, seeking some indication that account had been taken of *time* as a factor in the construction of concrete pavement; some recognition of the fact that the *finishing* of concrete pavements functions usefully beyond just the production of a smooth surface; or some admission that the essential *pavement qualities* are not *inherent* in the concrete as such, but must be *imparted* after the concrete has been placed. Candor compels the admission that no such evidence was found.

Current paving literature, as presented in our leading technical publications, is also barren of any such suggestions, but on the contrary, there seems to be a growing disposition, in public expression at least, to frown upon the spending of much time or labor on the concrete after placing. However, during the past year or two, actual practice, here and there, is recognizing the desirability of some more labor, if not much more time being used in finishing.

I have heretofore expressed the opinion that the chief difficulty in laying concrete pavement is not lack of knowledge regarding the making of *concrete*, but the lack of a definite conception of just what is necessary to be, or that may be, accomplished in the process of *finishing;* just what difficulties are to be met in the metamorphosis of the concrete into pavement, and the nature of the means to be adopted to overcome them; and a just appreciation of the characteristics of the material of construction. These things clearly and definitely stated would provide the basis for a consistent "practice," and constitute what might be called a "Theory of Construction."

The common practice in laying concrete pavement emphasizes that part of the process that precedes the placing of the concrete, and discourages or reduces to the minimum subsequent disturbance. But there are a number of actions and reactions, aside from setting, requiring time, that must take place in the concrete before it becomes pavement.

As an example, the water in a concrete mixture serves two distinct purposes, but it does not serve them concurrently. The quantity required for one is entirely too much for the other. The first service performed is

the lubrication of the mass, producing workability, the other is the hydration of the cement. It seems impossible to have ideal workability and the correct water-cement ratio in the concrete as it comes from the mixer, and in the absence of the *time factor*, the only alternative is to attempt to so control the water in the mix as to have a reasonable degree of workability without too great an excess for a reasonable approximation of the correct water-cement ratio. It seems that in the usual practice, about the best that can be done is to admit from 10 per cent to 15 per cent excess water.

Professor Abrams has said that workability is the only limitation that prevents the reduction of the cement and water in the batch to much lower limits than are now practicable.

The waste of cement through trying to procure workability and the correct water-cement ratio in the mixed concrete is perhaps the most serious criticism of the prevailing practice in laying concrete pavement.

This new method introducing the *time factor* in the *finishing* of the pavement, and establishing the correct water-cement ratio by quickly withdrawing and disposing of the excess water needed for workability, after that need has passed, avoids all conflict between "workability" and the "water-cement ratio," and secures the benefit that accrues from the reduction of the cement in the batch, as suggested by Professor Abrams.

This saving of cement is very material, as workability in pavement concrete admittedly requires from 10 to 15 per cent excess of water, and Professor Abrams further says that 13 per cent excess is equivalent to a loss of one-third of the cement in the batch. The saving in cement alone by the use of this method of laying concrete pavement would enable the building of hundreds of miles of additional roads each year, and there are numerous other advantages in its use.

The practical application of this *time factor—water-cement ratio* practice to the actual construction of the pavement slab develops certain important structural facts, conditions, and requirements, and affords the opportunity of observing certain relationships, and of calling attention to the apparent desirability of changing some of our definitions.

The following may be accepted as expressing in a way the "theory" of this new practice in laying concrete pavement:

> Concrete is a *plastic* structural material, manufactured at the site of the work. It is chiefly a matter of materials and mixtures, and any material delay in using it is prohibited.
>
> Concrete plus *time* and *labor* is a concrete *product*.
>
> When intelligent design and skilled workmanship have united in fashioning concrete into a useful thing, it ceases to be *concrete*, meaning thereby, a structural material.
>
> Concrete permitted to set in the condition in which it comes from the mixer, as is the case in other structures, does not possess to the necessary degree the qualities required in pavement: density, tensile and compressive strength, resistance to shock, to abrasive wear and to

the effects of changes in temperature and moisture content, or impermeability. These properties must be imparted to the slab after the concrete has been placed.

Making concrete and making concrete pavement are distinct operations. The former is a matter of materials and mixtures chiefly, while the latter is a matter of workmanship, purely. In the making of concrete, *motion* is the important factor rather than time. In the making of pavement, *time* rather than motion is the important factor. The impartation of the qualities that distinguish pavement from most other concrete structures, requires time not only for the necessary manipulation, but for certain chemical processes. The time so spent belongs to the making of *pavement*, and not to the making of *concrete*.

Density is a basic quality in pavement, and a prime factor in securing the other qualities.

Compressive strength is not just a convenient measuring stick for other qualities, but is a structural necessity.

Any method of finishing that impounds the excess water in the slab is a violation of the water-cement ratio theory, and affects both density and compressive strength.

The water in a concrete mixture serves two distinct purposes, but it does not serve them concurrently, and the quantity required for one is altogether too much for the other.

A film of water left on the surface when finishing results in a friable, abrasive surface, and finishing with a belt or similar appliance necessarily implies an appreciable quantity of water in or upon the surface.

Ceasing to work the surface at the stage when finishing tools are easily drawn over it, is depriving it of the most useful portion of the entire finishing process.

To produce a dense, hard, tough, non-abrasive, impervious skin coat or wearing surface, the surface mortar must be a rich mixture, and be worked until it becomes pasty and adhesive.

Horizontal force or movement,—floating, applied to the surface of the slab in connection with a rich dry mixture to extract the excess water, is more effective in *imparting* pavement quality than is vertical force—tamping.

Much tamping is harmful and is responsible for the present outcry against "overfinishing."

Smoothness of the surface of pavement is largely dependent upon two things; uniform distribution of the coarse aggregate, which can be attained only by using a maximum quantity, and a uniform and minimum water content at the time of final finish.

The total shrinkage in concrete pavements may be largely controlled by the judicious use of capillarity in removing the excess water.

The shrinkage that is incidental to setting may be largely overcome by proper attention to the absorption of the base, and by the long

continued use of the heavy floats, which constitutes a virtual retempering of the concrete.

A lean mixture has other advantages than just the saving of cement.

Surface cracking is caused by evaporation and early shrinkage in the surface portion of the slab before the concrete has attained strength to resist, and while the remainder of the slab still retains its excess water and original bulk.

Surface cracking is prevented and the upper portion of the slab made the strongest part of the pavement, by the application of a dry mixture of sand and cement which draws the water from the interior of the slab to the surface and keeps that at all times the wettest portion. Subsequent floating to evaporate this water also closes the voids within the slab. The application of the rich dry mixture being thoroughly incorporated, neutralizes the laitance and gives the pavement a proper wearing surface.

The extraction of the excess water in the slab by a rich mixture of cement and sand enables the constructor to establish the correct water-cement ratio at the precise time in the process of finishing that experience and judgment indicate is most beneficial.

In the light of the foregoing, I submit this definition of "concrete pavement":

Concrete pavement is concrete of good quality, in which certain inherent and disqualifying weaknesses and tendencies have been replaced with other and useful qualities not naturally resident (in any large degree) therein, through the application of skilled workmanship, under intelligent supervision, rendering it dense, resistant to the effects of changes in temperature, and in moisture content, to abrasive wear, to tensile and compressive stresses, and impermeable.

Concrete pavement is a structure built of concrete. The concrete goes into it in a plastic state, and when it hardens it becomes pavement of one sort or another depending upon the quality of workmanship applied during the interval between the placing of the concrete and the final manipulation. The *concrete* was a finished product and the erection of the structure begun, when the concrete began to be placed in the forms. It is at this point that the TIME FACTOR begins to function in the making of pavement.

As regards the making of concrete, Professor Abrams has reduced it to an exact science. He has pointed out how concrete of any predetermined strength may be made. But for the converting of the concrete into pavement no rules can be given. The behavior of the concrete under the constantly changing conditions surrounding its placement cannot be definitely forecast. Changes in base conditions, as to quality of soil, moisture content, compactness, absorption, gradient; in atmospheric conditions, as to temperature, humidity, circulation of air; time of year, time of day, cloudy or bright; and in materials, as to grading, water content, absorption, clean-

liness etc., occurring in every working day, and not overlooking the personnel of the contractor's organization, are in themselves sufficient to produce very marked differences in the quality of the pavement laid. For this reason skilled workmanship under trained supervision must be depended upon to interpret properly the reactions of the concrete to these changing conditions as they occur, and to apply the proper treatment to effectually control them in imparting pavement qualities.

I think it will be readily apparent that while the making of concrete is a science, the making of concrete pavement is an art.

While the enunciation of the water-cement ratio theory marked a new era in the making of concrete, up to this time its practical application to the problem of pavement construction, both in thought and in practice, has been confined to the mixing of the concrete. Its paramount usefulness in making pavement has not as yet been realized because of the failure to distinguish between the science of making concrete and the art of making concrete pavement, permitting time to function as a factor in this latter operation, and making it possible to establish the correct water-cement ratio after the concrete is placed.

In 1911 and 1912 we laid about 15 miles of plain 5-in. concrete pavement at Sioux City, Iowa. We have there a great variety of soils, ranging from heavy gumbo to a light wind drift which geologists call loess. When we began laying pavement, we knew nothing of the significance of these different soil types, but we soon learned from experience that with substantially the same methods, materials and men, we were getting a variety of qualities of pavement.

Our loess soils, which are highly absorptive, were giving us the best pavements. Considering the requirement in all specifications, including our own at that time, that the base should be thoroughly wet, how were we to account for these results?

Having had considerable trouble laying pavement on our gumbo soils, we tried making those bases absorptive. The improvement was undeniable, as results were approaching those on the loess soil.

We had learned the value of an absorptive base by 1914, and with this conception of its functioning, it was an easy deduction that more work on the surface while the concrete was still plastic would increase the density by closing the voids from which the water had been drawn into the base.

Attempting that, however, led to the discovery that the absorptive base was not functioning for the full thickness of the slab and that the upper portion still retained its excess water. This in turn led to a fuller appreciation of the function of the dry mix and the application of "sufficient dry mixture to absorb *all excess water*," raising it to the surface where it could be more readily expelled into the atmosphere. All this required an increasing amount of labor and time, which have since been distinguishing features of this method.

In 1919 we eliminated from our specifications the provision requiring the wetting of the base, and made it subject to the direction of the engi-

neer in charge, thus making it a matter of personal control for each case, which really had been the practice for some time already. We also inserted another provision prohibiting the laying of pavement on soggy or muddy bases, and requiring that such bases be lightly scarified to aid in drying them and increasing their absorption.

The effect of capillarity on the removal of excess water from the slab is very marked, and general practice aims to prevent it. The base is saturated to prevent its escape downward, and if that is not sufficient, it is frequently covered with tar paper. Escape upward by evaporation from the surface is prevented by finishing as quickly as possible and covering at once with wet burlap. Preventing the escape of free water would seem to be a serious violation of the water-cement ratio theory, and a complete subversion of the "time factor."

In our practice, in connection with the time factor, we find it of great benefit to encourage this escape of free water by capillarity, both downward and upward, downward by controlling the "suction" or absorption of the base, and upward by increasing capillary action through the application of a highly absorptive dressing, and the continuous floating of the surface enabling evaporation to the utmost.

By floating the concrete through the quaking stage, especially when a relatively large proportion of crushed rock aggregate is used, gravitation is permitted to settle the coarse aggregate more compactly, and this internal movement of the coarse aggregate closes the pores and small voids in the mortar portion of the concrete.

The quaking motions of concrete are horizontal, not vertical, and are easily imparted. We take advantage of this horizontal pliancy in "finishing," applying horizontal force or motion, rather than vertical, using the float rather than the tamper.

The value of the "time factor" lies chiefly in the opportunity it affords for the removal of the excess water, increasing the density of the slab, and producing an effective wearing surface.

Our theory and practice regarding the functioning of the base received definite confirmation in a paper by J. C. Pearson, published in the 1921 *Proceedings* of this Institute, on the "Shrinkage of Portland Cement Mortars," the results of a study of the subject as related to stucco coatings. Briefly summarizing his results after testing 200 slabs 1 in. thick, some poured in watertight, non-absorptive molds, and others in molds having absorptive bases:

"Volume changes, while the mortar was still plastic, in those poured in non-absorptive molds [corresponding to pouring pavement concrete on non-absorptive bases—T. H. J.] were too great to be considered typical of what happens in actual stucco coatings. Volume changes in those poured in absorptive molds [corresponding to pouring pavement concrete on absorptive bases—T. H. J.] of 0.02 per cent were exceptionally high." Also, "that the degree of wetness of the base has a very marked effect on the shrinkage taking effect after the mortar has set." Further, "that control of the

absorption in the base is the most vital factor in the control of shrinkage in both plastic and hardened mortars."

These conclusions had been worked out in the field, laying pavement slabs too thick to be affected throughout by the absorption of the base alone, and additional absorption had been provided to care for the greater depth of slab, in the application of a dry mixture of cement and sand to bring the remaining water to the surface where it could be liberated by evaporation.

This method, by *removing* the excess water in pavement concrete after it ceases to be needed for workability, rather than trying to mix the concrete sufficiently dry to secure the correct water-cement ratio, overcomes some of the basic troubles that are manifest to all who are in close touch with actual construction.

In placing concrete, because of the difference in the specific gravity of the materials, water rises to the surface bringing with it air and laitance. A part of the water, with its burdens, is entrained underneath the pieces of coarse aggregate, causing voids and a poor bond. Water continues to rise until stopped by the setting of the mortar.

In the ordinary manner of finishing pavements, these conditions result in an increasing water-cement ratio from the bottom upward, with a consequent decrease in density and strength in the same direction, and with a surface coating of laitance.

In the method of finishing pavements that I am attempting to describe, in which the "time factor" is a characteristic feature, advantage is taken of the tendency of free water to escape from the slab. Capillarity is capitalized, by providing an absorptive base and a highly absorptive surface treatment not only to intercept the water that voluntarily rises to the surface but to attract through capillarity all free water within the slab. Within from fifteen to twenty-five minutes after placing a fairly wet mixed concrete, it has been leveled with shovels, floated and crowned with heavy floats, rolled and belted to dispose of surface water, dressed with a dry mixture of equal parts of cement and sand at the rate of one sack of cement to approximately 20 sq. yd. of pavement, the quantity depending upon a number of conditions, and is being belted and floated with long-handled heavy oak floats. This floating is continued until the surface mortar becomes pasty and adhesive.

This manipulation of the pavement with the belt and heavy floats, beginning very shortly after applying the dry mix, when the mortar is at its most plastic stage, anneals the slab to its full depth, and being able to finish the surface after the removal of the excess water from below, and the positive preclusion of further moistening of the surface, enables the establishing of the correct water-cement ratio and the impartation to the entire slab from the top downward of a full measure of the essential qualities of good pavement and a wearing surface that is free from laitance—hard, dense, non-abrasive and impervious.

DISCUSSION.

Mr Gilkey. HERBERT J. GILKEY (*By Letter*).—Mr. Johnson's paper is noteworthy in several respects. It shows a recognition and, what appears to be, a rational solution of problems that are today unrecognized and unsolved by the great majority of concrete operators. It is little short of remarkable that Mr. Johnson should have developed so excellent a technique in a period prior to a general knowledge that excess water was objectionable and laitance preventable.

His distinction between the manufacture of the concrete and the making of the pavement is valid and commendable. A general appreciation of the fact that there is something more to the laying of a pavement or the pouring of a building than merely mixing the concrete and dumping it into the forms, is probably the next great truth that must be hammered home.

Concrete must be properly placed. Then it must be cured, a process that requires moisture and temperature applied simultaneously. None of these is adequate without the others. Mr. Lord, before the convention * has admirably stressed all of these points, with special emphasis upon temperature and moisture. The writer has demonstrated,[1] in common with many others, the need of adequate moist curing. Mr. Johnson in this paper places commendable stress upon the technique of proper placing.

Good concrete of proper cement and water content, suitable aggregate, right workability and adequate mixing are pre-requisites, but as soon as that concrete is deposited, a separation starts that continues both during and following the tamping period. Solids settle and water rises. The top surface becomes moist or even very wet. This process of sedimentation continues until initial set takes place. Thus the top concrete is very wet before setting occurs and therefore very weak. This "Water Gain"[2] produces weakness and brings laitance to the upper surface. Thus the ordinary methods of making and finishing most concrete surfaces are conducive to weakness in compression, tension and abrasion. The first is of least interest in this connection. The tensile weakness promotes hair cracking which is augmented by the laitance. The rising water brings to the surface the finest particles of the cement and sand. It also brings clay and any other very fine or colloidal matter present. Very fine materials shrink greatly upon drying. Thus we have a surface subject to abnormal shrinkage and low in tensile strength. Is it any wonder that hair cracks and dusting are the rule?

Mr. Johnson's dry absorbent subgrade sounds very reasonable. It has been amply proved that extraction of excess water shortly after mixing

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* "Notes on Concrete—Wacker Drive, Chicago," p. 28.

[1] "Curing Conditions of Mortars and Concretes," *Proceedings* A. C. I., 1926, pp. 395-436. "Finishing and Curing of Concrete Roads—Discussion," *Proceedings* Am. Soc. C. E., Jan., 1927, pp. 79-93.

[2] For more detailed consideration of this, see *Engineering News-Record*, Feb. 10, 1927, "Water Gain and Allied Phenomena in Concrete Work."

improves the quality of the concrete in the same manner as the initial use of less water.[1]

Without detracting from the probable virtue of the dry subgrade the writer desires to especially commend the surface treatment that Mr. Johnson uses. Working dry sand and cement into the top surface, by the continued troweling for a period of many moments undoubtedly produce a surface of maximum strength, density and hardness against abrasion, as well as a much stronger interior by virtue of the maximum water extraction that has occurred. There is reason to suspect that the extra working assists the cement in hydrating, but even if this were not true, the virtue of the method is established.

The surface treatment is readily applicable to floors and sidewalks, and has sometimes been applied to them. The quality of the concrete in many forms of construction could be materially improved by an application of similar treatment. It is not improbable that a few years' time will see Mr. Johnson's "dry subgrade idea" applied to regular building construction. Forms could be lined with absorbent material that would extract considerable of the excess water from the freshly placed concrete.

The writer has produced an increase of 10 to 20 per cent in the 28-day standard strength of concrete and mortar by troweling in dry cement as fast as the moisture rose to the surface. This made the top the strongest, instead of the weakest part of the specimen and extracted excess water from well down into the body of it.

Mr. Johnson's methods have enough of demonstrated virtue to commend them to the thoughtful consideration of us all. There is probably little concrete "poured" that could not be greatly improved by developing a technique of placing similar to that which Mr. Johnson has successfully applied to paving work.

Mr. Chapman.

CLOYD M. CHAPMAN.—I was interested in the statement Mr. Johnson made that his method of treatment resulted in getting exactly the right water-cement ratio. What do you consider the right water-cement ratio in a concrete pavement? What water-cement ratio do you get?

Mr. Johnson.

T. H. JOHNSON.—If I made it as positive as that, it was unintentional, because it appears to me that I put in the qualifying word there "approximately" in the first mention as to the results of this mixture, but the result we get is just this: Within about 7 or 8 min. after we have placed this concrete, and applied the dry mixture, our finishers get out on the surface of the concrete itself without bridges or anything of that sort, and one can see their shoe soles, so that we have extracted the water to the extent that the concrete is not disturbed by an ordinary man walking on it. It has not set up, it has simply dried out.

Mr. Lord.

A. R. LORD.—Is that before the dry finish goes on?

Mr. Johnson.

MR. JOHNSON.—At that time the slab has been floated with long-handled oak floats, rolled and belted, and the dry finish applied, but not floated.

[1] For recent allusions to this see papers of John J. Earley, W. Paul Eddy, and F. F. Longley before this convention.

Mr. Upson. MR. UPSON.—How long after the pouring is that floating begun?

Mr. Johnson. MR. JOHNSON.—Almost immediately. The floater works right up to where the shovelers have levelled it off. After the preliminary floating, it is rolled and belted.

Mr. Upson. MR. UPSON.—And the dry mixture is not applied immediately?

Mr. Johnson. MR. JOHNSON.—No. We do as much work on this pavement before applying the dry mixture as the ordinary method of finishing requires when it calls the pavement completed. At that stage we apply the dry mix which pulls out of the slab water sufficient to make it a thin, watery grout; then we get on it with a belt and long-handled floats and work it until we have evaporated all that water.

Mr. Upson. MR. UPSON.—Is that dry mix the same as your aggregate?

Mr. Johnson. MR. JOHNSON.—It is a 50-50 mixture of cement, and sand passing a No. 8 screen.

A Member. A MEMBER.—How thick is that coat?

Mr. Johnson. MR. JOHNSON.—It has no definite thickness; 2 cu. ft. spread over 20 square yd. would not add any appreciable depth to the slab, but it does do something when it comes to lifting out the water. One sack of cement to 20 sq. yd. is not a hard and fast rule; the amount that should be used depends upon a number of different conditions, not only the consistency of the concrete but the atmospheric conditions and the condition of the base.

A Member. A MEMBER.—What is the advantage of a lightly-rolled sub-base?

Mr. Johnson. MR. JOHNSON.—It leaves it absorptive so that it takes a part of the water out of the slab, and less likely to be heaved by the frost.

A Member. A MEMBER.—If the sub-base was more thoroughly rolled, would it deprive it of its absorptive qualities?

Mr. Johnson. MR. JOHNSON.—Yes, to a certain extent. If thoroughly rolled, or if compacted very much beyond the natural soil condition, nature begins immediately to restore the natural condition. One of the chief disadvantages of a dense base is subsequent frost action.

Mr. Lord. MR. LORD.—How wet was this concrete? What would it be in terms of slump? What curing, if any, did the slab receive after the work had been done upon it? Was any damp sand, calcium chloride or anything of that sort used? What did you do in the case of rain, how long did you have to wait before you could go ahead?

Mr. Johnson. MR. JOHNSON.—We wait longer, as a rule, than the contractor likes. Now, just state those questions in order once more.

Mr. Lord. MR. LORD.—First, what, in terms of slump, how wet is this concrete?

Mr. Johnson. MR. JOHNSON.—Consistency is not as important with this method. Slump tests and the like may be omitted, because the excess water is to be removed as soon as the pavement is crowned. Part of it goes into the base and the remainder comes out with the application of the dry mixture. Instead of covering it with a wet towel to keep it wet, we keep it constantly stirred with the heavy floats and belts to evaporate the water. That keeps the top of the slab the wettest portion all the time, until finishing is complete, and when that is the case the surface will not crack.

MR. LORD.—There cannot be more than a 2 or 3-in. slump. Most contractors would say it was too stiff if they had to put it in a reinforced-concrete building. Mr. Lord.

MR. JOHNSON.—We are considering only concrete for pavement. A thoroughly workable concrete is discharged from the mixer and it is not stiffened up until the slab is properly shaped and the need for workability has ceased. Mr. Johnson.

R. W. MCBETH.—I observed the Johnson concrete being placed in 1916, and I would say that it had a 5 or 6-in. slump. Mr. McBeth.

MR. LORD.—The other question was about curing. Mr. Lord.

MR. JOHNSON.—We usually sprinkle the pavement for 6, 7 or 8 days after finishing. Mr. Johnson.

STEPHEN FLAM.—What part of the excess water is absorbed by the base? Mr. Flam.

MR. JOHNSON.—I do not know, a third of the way up the slab perhaps. Mr. Johnson.

S. C. MCKEE.—What special treatment is given this base? Do you attempt to get uniformity there? Mr. McKee.

MR. JOHNSON.—In the base? Mr. Johnson.

MR. MCKEE.—Yes. Mr. McKee.

MR. JOHNSON.—We roll it with a 6-ton roller, and aim to have as nearly uniform density as is possible. Mr. Johnson.

MR. MCKEE.—Do you add any granular material to your subgrade? Mr. McKee.

MR. JOHNSON.—No, but we aim to have the soil of the base in an absorptive condition. Frequently when we are working on a gumbo soil, we scarify the base lightly with an old-fashioned spike-tooth harrow and then place the concrete. Mr. Johnson.

MR. MCKEE.—In other words, if you have soft conditions, it tends to work the concrete more in some places than others to get the absorption. Mr. McKee.

MR. JOHNSON.—That is it, and that is one of the advantages of this method of finishing pavements; if there is an exceptionally wet batch dropped, or if the base has become saturated when the mixer has stood, there is an area which does not dry up like the other. In such cases additional dry mixture with additional work may be applied, and the slab be finished, having a uniform water content. Mr. Johnson.

MR. MCKEE.—Is the cost of the subgrade treatment included in the price of the pavement? Mr. McKee.

MR. JOHNSON.—Yes, and the prices given included 6-in. of excavation that is replaced by the pavement. Mr. Johnson.

MR. MCKEE.—What provision did you make for expansion? Mr. McKee.

MR. JOHNSON.—We put expansion joints every 50 ft. transversely. Mr. Johnson.

MR. FLAM.—What would happen if you would not put the dry cement and sand on top? Mr. Flam.

MR. JOHNSON.—The benefits of this method of finishing would be lost. Mr. Johnson.

MR. FLAM.—Would not the bottom absorb it ultimately? Mr. Flam.

MR. JOHNSON.—No. The greater specific gravity of the aggregate tends to force the water upward and the absorption of the base is thus Mr. Johnson.

balanced. By applying the dry mixture the water is at once drawn to the surface where it can be more readily evaporated.

Mr. Lord. MR. LORD.—I think this method is acceptable to engineers generally who are familiar with the proportioning of concrete, but I cannot see where there can be any saving except in the price of materials and labor. On Wacker drive we used a somewhat leaner mix with from ½ in. to 0 slump and we tamped it, and we had a strength commonly of 6,500 to 7,000 lb. at 28 days, which is about 40 per cent better, and the amount of work done was much less than you describe. Both methods are acceptable, I think.

Mr. Mellos. JOHN S. MELLOS.—Did you make any attempt to measure the quantity of water you used?

Mr. Johnson. MR. JOHNSON.—Yes.

Mr. Mellos. MR. MELLOS.—Does the work with the floats tend to drive aggregate away from the surface toward the bottom of the mix?

Mr. Johnson. MR. JOHNSON.—No. There is a fair sample there (indicating sample of the pavement) if you want to see what the results are.

Mr. Earley. MR. EARLEY.—For fear there might be an impression in the minds of some members that the processes which Mr. Johnson described are entirely heretical, I would like to correlate them briefly with the technique of plastering, which, of course, has made its own history. Let me direct your minds to a comparison between the absorptive base of soil and a green wall. Both of these are absorptive bases. In road construction the concrete is applied over this absorptive sub-soil. In a plastering operation the brown coat is applied to the absorptive base. Part of the water from each slab is drawn into the absorptive base and part of the water in each case is dissipated into the air. This dissipation is affected in time and by manipulation. Mr. Johnson described a float which I understand is made from a rather heavy piece of wood which is dragged over the surface of the concrete slab until density and stiffness and relative dryness is procured. This is analogous, in my mind, in every respect to the operation of sand coating in plastering, which is a manipulation designed to close the pores of the plaster slab while the water is being dissipated from it by means of manipulation. It also is floated by a wooden float until the mass is compacted and until the surface is relatively dry.

A Member. A MEMBER.—Mr. Johnson, why do you omit reinforcing?

Mr. Johnson. MR. JOHNSON.—Because nobody has convinced us of its necessity or desirability. Our pavements do not crack more than the pavements that are reinforced, in fact not so much. We have pavements 3 or 4 years old, 50 ft. wide, without center joints, that have no cracks in them. I have a letter regarding a pavement of 18,000 yd. laid in 1924, part of it on the main highway through the town, and all 60 ft. wide from the president of one of the town's banks saying there is no crack in it.

A Member. A MEMBER.—What part of the country is that in?

Mr. Johnson. MR. JOHNSON.—In northern Nebraska. We have no objection to reinforcing; anybody can have it that wants it.

Time as a Factor in Making Portland Cement Stucco.

By John J. Earley.*

This paper is designed merely to point out certain phases in the making of old-fashioned stucco, wherein time was an important factor, and then to relate these more or less definitely to similar phases in the making of modern portland cement stucco and concrete. Before writing it some old and dear friends were consulted, Viollet le Duc, Vitruivuus Pollio, Vasari, Merrifield, Bankart and Miller. They told how stucco was made with mud in Egypt and how it was kept from cracking, how Greeks applied exceedingly thin coatings of very hard stucco to coarse-grained stone working into the stucco density and hardness and a finish equal to a polish; how the master craftsmen of each age did their work and they whispered secret formulas, weird and wonderful mixtures of gall and blood, eggs and milk with more familiar lime and sand, gently insisting all the while that to this knowledge must be added time and the greatest of care.

We on the other hand pay little heed to time and the greatest of care, our care is more for the strength of our materials than for any other property, but the old craftsmen paid equal attention to workability, density, movement, uniformity, and appearance. They recorded what to do that the materials might be strong and enduring, what to do to make them dense and waterproof, how to keep out cracks, which naturally should appear when the water dries away, how to make each mix like every other mix, and how to make the surface pleasing to the eye.

Perhaps, some day, when these other properties receive equal recognition with strength, other branches of technique, having to do with them, will be discussed and there will be applied to the making of portland cement stucco and concrete all the wisdom of the past and time and the greatest of care.

Here is the translation of a very old description of the tempering of lime for stucco (Joseph Gwilt of Vetruivuus Pollio): "The lime shall be of the best quality and tempered a long time before it is required for use, so that if any of it be not burnt enough the length of time employed in slaking it may bring the whole mass to the same consistence. If the lime be not thoroughly slaked but used fresh, it will when spread throw out blisters from the crude particles it contains, which, in execution, break and destroy the smoothness of the stucco. When slaking is properly conducted and care taken in the preparation of the materials a hatchet is used similar to that with which timber is hewn, and the lime is to be chopped with it as it lies in the heap. If the hatchet strikes upon lumps the lime is not sufficiently slaked and when the iron of the instrument is drawn out

* Architectural sculptor, Washington, D. C.

dry and clean it shows that the lime is poor and weak, but if, when extracted the iron exhibits a glutenous substance adhering to it, that not only indicates the richness and thorough slaking of the lime but also shows that it is well tempered."

It is easy to read in this description the recognition given by craftsmen of other days to the factor of time. The first thought expressed in it was of dry lump lime of the best quality which subsequently was tempered or mixed with water. Here is the first movement of time in this specification the relation in succession of two states in a series of states, a dry state and a wet state. The second relation described in the sequence is that between material which is simply wet and the same material in a more intimate relation with water.

In contrast to the first relation, which moved quickly, the second relation progresses slowly beginning with the finest and extending to the coarsest particles. It is described in the quotation as "A long time before it is required for use." It is clear, then, that the intention was to free the lime, in time and by manipulation, from lumps and to "Bring the whole mass to the same consistence." But more particularly the wish was to develop in the lime a "glutenous substance," which was regarded as the measure of its richness and of the skill of the workman.

We know that the most desirable properties of stucco and of concrete are to be had only by proper manipulation at the proper time of all the ingredients. We know that the ideal conditions for the application of stucco and for the placing and molding of concrete are not those in which the greatest density, impermeability and strength are developed. It is picturesque and interesting to turn to the old masters for corroboration of this knowledge but it is unnecessary because our own experiences force it upon us. We know that something must be done to these materials at a certain time to change them from one phase to another. They must have energy expended on them, they must have work done on them and that in time and through the common medium—water.

Considering the circumstances in which we live and the intimate relation of water to us and to the things around us, its use for the control of materials might almost be regarded as an intuitive art, and indeed one may well believe this in some considerable measure to be true, for the most ancient arts are those in which water is made to combine with other material or is used as a medium for the application of energy to other materials, that is for the working of them. The art of the plasterer is of the oldest of these. In it he learned the physical characteristics of his materials and the laws which govern them, intuitively conforming to them long before he was able to analyze the principles or to record the laws, and among the first principles which he recognized were time and the greatest of care. By this it is meant that he learned to recognize a succession of phases in his material and to apply carefully and properly such force or energy, such work as was necessary to influence the character of the materials in each phase through which they passed to a finished state. It is characteristic of

materials which form intimate relations with water that little energy is required to influence them. Even the fullest range of influence affected by extremes of energy, such as would change ice to steam, is easy when compared to what would be required to change stone to glass or ore to steel. This characteristic affords great facility in the working of these materials and permits human power coupled with time to do all that is necessary to plastic materials.

When the old specification described "glutenous substance" in lime it pointed out that state of material developed by hydration which gave to the lime its workability and its strength. A similar state is developed by hydration in portland cement but not of the same substance nor with the same time relation and, therefore, not subject to the same technique. Nevertheless, it is recognized by craftsmen that the workability of cement stucco and concrete may be improved by allowing the materials to stand after mixing and before use. It is only necessary for illustration to suggest one or two of many well-known examples of this practice. A mason preparing cement mortar for use in pointing mixes the materials for a long time and then allows them to wait while he prepares for the work and a plasterer experienced and skillful with portland cement stucco mixes his material in a mixing machine for a considerable time, possibly ten to fifteen minutes, and then allows it to stand. These craftsmen know that materials so treated will work better and spread further than materials mixed by hand and applied immediately. They know workable or "fat" mortars are those which retain enough water when they are brought into contact with other and dryer materials to preserve a degree of plasticity sufficient for forming and finishing, and that harsh or "short" mortars are those which lose water easily and rapidly when they are brought into contact with other and drier materials making them difficult to form and still more difficult to finish, they also know that other desirable properties can be put into the materials by proper manipulation at the right time.

Data tend to show that the strength of concrete is improved when it is allowed to stand in a wetted state for a reasonable time and when mixing water in excess of the amount required for hydration is removed before hardening. In addition to this, experience indicates that a reasonable time lies somewhere between two and four hours, depending upon modifying circumstances such as temperature, humidity and the nature of the cement. The suggestion that concrete as well as stucco may be mixed with an excess of water to be extracted, after the concrete has been placed in forms, for the improvement of its strength should not be received with surprise. The similarity between stucco and concrete is common knowledge and experienced plasterers know that while nothing good ever comes of a sloppy mix, yet, for the sake of workability the consistency of mortar should be as soft as is practical relying on the absorption of the base or the undercoats to extract all excess water and thereby to compact and strengthen the stucco. The whole technique for applying stucco in several coats is based

upon this principle, which is also a factor of time, a relation between two successive states. In one state the materials wet and partly hydrated containing an excess of water designed to act as a medium for the application of energy, for workability, are free, uncontained; in another state the materials more fully hydrated, without excess water, and compacted for the greatest strength, are restrained in forms.

The technical control of the consistency of cement stucco at the proper times by tempering with water and by subsequently extracting the water, is an application of the "water-cement ratio" but in two steps instead of one. The application of stucco to vertical and horizontal surfaces requires the water content to be held within very close limits at critical times. At one time the stucco must be sufficiently wet to make the mass workable but not wet enough to make it slide or slip, at another time the stucco must be dry enough to be a stable mass but not dry enough to impair seriously the hydration of the cement. So, even within such close limits as these there are two water relations, one proper to the plastic state, that in which stucco must be worked, and the other to the solid state, that in which it hardens or sets.

The most difficult application of technique in the making of stucco is that which belongs to the phase which is next to the last phase, or to the time which relates the finished state of the material, that in which it has achieved its final set, to the state just before that in which it has been applied but has not lost all the free water. In this phase time is an important factor and the need for great care in pressing, because the time which measures this sequence is that in which much must be done of what can be done to prevent crazing.

The most perfect control that has in my opinion ever been exercised in this particular phase of stucco was exercised by the Greeks when they applied to their buildings with cut stone a thin, hard polished coat of lime and marble dust in preparation for a painted finish. To appreciate the difficulties of this application it is necessary to remember that the stone base was practically without absorption and that the free water in the stucco was lost principally by evaporation. The stucco is thought to have been a mixture of about two parts of lime putty and five parts of finely-ground marble dust and enough water to make the mass plastic. The materials were beaten together with much labor by men with sticks (Bankart). They were applied in two or three very thin coats and were carefully trowelled and retrowelled at frequent intervals, gradually compressing and closing the voids which undoubtedly developed during the shrinkage of the mass due to the loss of water.

In white finishing plastered surfaces the same principles are applied as were described above with the following notable exception: modern white finish is usually applied over an absorptive base. This base extracts free water from the finish more rapidly than it could be lost into the air. The action stiffens the mass and permits it to be finished with less effort but also with less perfection than the ancient stucco of the Greeks. It is

questionable whether many of our craftsmen now know how to control this particular phase of stucco.

Modern portland cement stucco although it seldom receives a smoothly trowelled finish is nevertheless subject to the same laws. Our best practice indicates for the minimization of shrinkage and crazing either a sandy mix applied over an absorptive base which will extract the free water within a short time, or a richer and more plastic mix applied in a thin coat and worked so as to compress it and to close craze cracks as they form. The final working should be done when the stucco is relatively dry. This technique is supported not only by the history and tradition of the plasterer's art and by present practice but by data such as were published in Technologic Paper No. 70 of the Bureau of Standards, special attention being directed to the history of panel No. 15.

It is not always easy to recognize in the mixing and pouring of concrete into forms the same principles which underly the mixing and application of stucco with hawk and trowel. Yet, the general characteristics of the material are the same and the general laws which govern them may be expected to be the same. Both concrete and stucco are composed of an inert aggregate bound together with some cementitious material and water and worked while soft with water as the medium. A complete analogy cannot at this time be drawn between concrete and stucco, because some phases of stucco do not match with corresponding phases of concrete and some characteristics of concrete, although similar to those of stucco are not generally recognized. But it can be suggested to those who wish to achieve the best technique for making portland cement stucco and concrete that a careful study be made of ancient and modern methods for making stucco whether with mud, lime or cement. Such a study should be sufficiently broad to enable the investigator to recognize in kind and, in a degree, relation between the different states of any material and between similar states or phases in different materials.

The suggestion in this paper that excess water may be added to stucco and concrete for the sake of mobility conditioned upon its removal before the time of hardening must not be misunderstood. It is not intended to be a license for the general use of wet mixes but rather a privilege applicable only to intelligent and conscientious craftsmen which may be exercised under the moral obligation to restore in time the proper water-cement relation for strength. Nor must the mention of two water relations be understood to mean that there are two water-cement relations properly applicable to stucco and concrete at the same time. There is but one water-cement relation for strength and that occurs in the materials when in place and at the time of hardening. The other water relation is a water to surface relation which includes both the surface of the aggregate and the cement and which has to do with the mobility of the mass and not with its strength.

The technology, therefore, of control by means of two water relations indicates that the water relation at the time of hardening should be to the

cement for strength and that the water relation at the time of mixing should be to surface and for workability. These relations are not fixed by any law and should be designed for every use of stucco and concrete. To illustrate this point let us assume that for a certain purpose we have designed two batches of concrete of equal strength, one of which is to be cast in an absorptive mold such as a sand mold and the other in a non-absorptive mold such as a metal one.

Let us also assume that both molds are of equally maximum complexity. It is reasonable to design the consistency of the concrete of both batches in the molds at the time of hardening to be that of concrete with a water-cement relation equal to 5 gal. of water to each bag of cement and with little or no slump, and it is reasonable to design a water content of 8 gal. for the same batch for placement in a sand mold when experience teaches the craftsman that the rate and volume of absorption in the mold will extract 3 gal. of water before hardening. On the other hand, such a design would be unreasonable for the plastic state of the batch to be placed in the metal mold and it might better be replaced by a design providing for the mixing of part of this batch to a consistency equal to that of concrete mixed with 6 gal. of water to the bag of cement and with a slump 4 in.; and for the mixing of the remainder of the batch to a consistency equaling that of concrete made with 4 gal. of water to a bag of cement and with no slump, and for the mingling of both parts by the craftsman while filling the mold in such a manner as to produce the predetermined consistency for strength.

It is true that there is not much scientific information available on this subject but data are accumulating and each development in research confirms the experience of craftsmen both ancient and modern accenting with them the need for time and the greatest of care.

Time as a Factor in Making Concrete—Cast in Sand Molds.

By W. Paul Eddy.*

The application of the water-cement ratio theory in sand mold casting in the Onondaga Litholite Co. plant is as follows:

We use 8 gal. of water to one bag of cement.

Our concrete mixture is one part cement to three and one-half parts of marble aggregate.

The marble is crushed at our plant, very carefully screened and graded to three different sizes—the largest of which will pass a ⅜-in. mesh.

There is no guesswork about how much water is used to the batch.

The water is drawn from a tank provided with a gage which shows the contents of the tank at any level. We are therefore able to draw off the desired quantity of water accurately.

The three different-sized marble aggregates are weighed separately in a hopper before being deposited into the mixer. The cement is added and then the water.

The mixer holds 1 cu. yd.

We mix this thoroughly for about four minutes, then transfer the contents to a subsidiary mixer or agitator in which it is continuously agitated or mixed until the last of the batch is poured. This entire operation takes about 10 min.

It would appear from data that we are familiar with, that the accepted reason for using this excess of water in sand-mold casting is the one of having the mix flow into the complicated parts of the mold. There is another reason why we use this excess water. We find that it is necessary to have the quantity of water as stated above in the mix in order to obtain the right consistency for pouring from agitator to mold.

A slightly drier mix might be used if the entire batch could be deposited by continuous pouring into one mold.

Very few molds require the contents of one batch, therefore the valve must be closed while the agitator is being moved to the next mold. This short time allows the mixture to clog in the spout and considerable time and mix is lost before actual pouring can start again. That is expensive and therefore not practical. We find that the proper mix for all practical purposes is as described above.

This ratio of water to cement in our mix does not make a liquid mass that flows freely, but makes a pasty mass that flows or rolls along in a sluggish manner in the mold at a grade of 1 in. to the lineal foot.

* Onondaga Litholite Co., Syracuse, N. Y.

While this amount of water that we use may be considered by some as excessive, we have never worried about this excess of water in the mix as it filters through the sand before the actual setting of cement begins. We have always believed that the excess water filtering away through the sand does not carry with it any of the cement particles because the sand is used over and over again, in fact hundreds of times, and the sand does not show any signs of solidification nor does it change in any of its plastic qualities for molding. In other words, when the stone is lifted out of the sand the walls of the mold are still soft and do not show any signs of hardening caused by cement mixture. Twenty minutes from the time cast is poured enough water has filtered through the sand so that the mass has settled and become firm enough to trowel.

We find that the cast stone made by this process requires 48 to 72 hours' curing in the sand before it can be handled and piled.

To prevent too rapid curing the sand adhering to the cast when taken from the mold is left on for at least two weeks. At this time the stone is hard enough to turn over to the finishing department.

The concrete we make by this method tests very high in compression—that is, between 5,000 and 6,000 lb.—and is low in absorption—tests show as low as 3 per cent and never higher than 6 per cent, and passes all building code requirements by such a wide margin of safety that we believe that we have reached almost the ideal in strength. We are convinced by the results of these compression tests that we really do observe very closely the ideal water-cement ratio.

The charts show that the maximum compression test of 6,000 lb. can only be obtained by a ratio of 3½ gal. of water to 94 lb. of cement.

As we get very close to this maximum strength in tests, it would almost conclusively prove that we lose enough water through filtration to account for the difference between 8 gal. and 3½ gal.

Time as a Factor in Making Concrete—Centrifugal Pipe Manufacture.

By F. F. Longley.*

In the manufacture of centrifugal pipe, the manipulation of the concrete immediately after mixing, is done in a way that results in a very low water ratio at the time effective hydration of the cement and setting of the concrete commences.

In making pipe by this process, the concrete or mortar is mixed and then placed within the mold which is whirled at a fairly high rate of speed. The centrifugal force acting upon the plastic mixture is like the force of gravity greatly intensified. The compacting of the solid particles of the mixture is aided, too, by the vibration which is an inevitable feature of the whirling process.

Since the water has lower specific gravity than any other part of the mixture, all of it which is squeezed out of the mixture as a result of its intensive consolidation, comes to the interior surface of the pipe wall, newly formed and continually rotating. The whirling action continues long enough for the materials to become well consolidated, and for the excess water coming to the inner surface of the pipe, to be removed. At this stage, the water remaining in the concrete is only the small amount required to fill the low percentage voids of the highly-compacted material.

All of this takes place in a matter of minutes after the cement and water are first brought into contact with each other and before it is possible for hydration of the cement to proceed to any observable degree.

It hardly need be explained that a mixture put into the rotating mold has to be of such a consistency that it will flow readily, in order that it shall distribute itself uniformly within the mold and produce a pipe wall of uniform thickness. The outside surface of the pipe is, of course, shaped by the metal of the mold. The inside of the centrifugal pipe is not formed against any surface, but its true cylindrical shape depends upon the ability of the freshly mixed concrete to distribute itself, by virtue of the rotating movement, uniformly throughout the pipe wall.

The following figures observed during the manufacture of 24-in. Lock Joint centrifugal pipe are typical of the water quantities used. The mixture in this case was a 1:2 mortar with 4¾ gal. of water per bag of cement. This does not represent a very high water ratio—only about 62 per cent—but it gives a mixture which flows easily into and about the rotating mold. In the case here referred to the amount of excess water removed during rotation was 2 2/3 gal. for each bag of cement in the mixture. This removal of water corresponds to what happens to a concrete of

* Lock Joint Pipe Co., Ampere, N. J.

low relative consistency—that is, fairly dry—when it is tamped persistently enough to bring excess water to the surface.

Thus, at the time when the centrifugal pipe wall has become sufficiently consolidated to retain permanently its shape against the interior of the mold, and the whirling is stopped and the pipe in the mold removed to the curing yard, the amount of water in the pipe wall is 2 gal. per bag of cement. This corresponds to a water ratio of 0.27.

It is commonly stated that 2½ gal. of water per bag of cement is sufficient or somewhat more than sufficient for the hydration of the cement. It thus appears that the quantity of water in the pipe as it is set away to cure is just about the quantity required for hydration with no measurable excess.

The fact that the consolidation of the material and the removal of excess water have proceeded to a point where the pipe wall retains its form permanently, and all within a matter of a few minutes, during which it is not reasonable to assume that the hardening is in any degree due to hydration, is virtually equivalent to saying the mixture in this stage has zero slump.

According to the arguments that have been used in connection with the water ratio theory, this low water ratio and zero slump should give a most excellent concrete. This we know is true. Concrete made in this way has a high specific gravity and a high degree of imperviousness; also a high crushing strength.

From the above remarks, the inference might possibly be drawn that centrifugal concrete is the best concrete for making pipe, and it *certainly* is for some conditions. We recognize the high qualities referred to, but although we are continually engaged in the manufacture of centrifugal pipe and the construction of successful pressure pipe lines, we would not venture to claim centrifugal pipe the best for, or even suited to, all conditions. In the wide range of conditions met with in the design of pressure pipe lines, considerations more insistent and more dominating than compressive strength and density are frequently met with and must be reckoned in the design. In our business, we have found it a safe and sound policy to limit the use of centrifugal pipe to the lower range of heads and to the smaller range of sizes.

Effects of Time Before Placement in Concrete Pipe Manufacture.

By George D. Chandler.*

There is probably no branch of the concrete industry where the application of scientific principles can be of more value than in the production of machine made concrete pipe, and there is probably no class of concrete concerning which so little is known as the very dry mix material which the pipe manufacturer compacts into the desired form by the use of packer head, tamper or centrifugal mechanism. During the past year two conditions apparently brought about by the condition of the concrete previous to the placement have been brought to our attention.

The first had to do with the problem of obtaining a satisfactory product as regards strength and soundness, using a tamping machine under severe winter conditions. We were operating a Quinn machine producing reinforced pipe in sizes 24 in. to 54 in. in diameter. We were using a steam-heated aggregate with a fineness modulus of approximately 4.5, a 1: 3 mix put through a 14-ft. Blystone mixer, with a 3½ to 4-min. mixing time. The pipe came from the machine apparently perfect, were placed in the kiln and the jackets removed. About 3 hours after the kiln was closed the steam was turned on and the kiln maintained with saturated atmosphere at approximately 110 deg. temperature for 48 hours.

Upon opening the kiln we would find from 10 per cent to 70 per cent of the finished product contained circumferential cracks close to the center of the mass in varying degrees of severity. Some might be merely an inch or two long, barely penetrating the pipe and some extending completely around and completely through the shell. These cracks were only a few thousandths of an inch wide, scarcely perceptible when the product was dry but very apparent when wet.

We felt from the nature of the cracks that they were due to a volumetric change in the pipe mass that stressed the reinforcing steel in compression to a point where the tensile opposing tensile stress on the concrete was beyond the strength of the material. Not knowing enough about the behavior of concrete to analyze our problem we were forced to rely on cut and try methods to overcome this difficulty.

The possibility of a sudden temperature change caused by opening kiln doors and allowing air at freezing to rush into the hot kilns being responsible was investigated. Various kiln temperatures from 70 to 150 deg. F. were used. We finally detailed a man to inspect the pipe hourly from the time it was made until it was cured and found the cracks were becoming visible between 5 and 10 hours after they were made.

* Superior Products Co., Detroit, Mich.

We next turned our attention to conditions inside the plant. We heated the reinforcing steel before placement with no effect—we raised the temperature of the kilns during loading—we varied the grading of the aggregate, tried a different cement, and varied the water content of the mix within the somewhat narrow limits that the machine would handle. None of these attempts remedied the situation.

We did find that reducing the temperature of the mix as it went to the machine reduced the amount of defective product. This entailed cutting off the steam in our aggregate hoppers and introduced the danger of getting lumps of frozen material into the completed pipe.

In the course of our investigation another possible remedy appeared. Occasionally due to manufacturing defects we found that a pipe will reach the kiln in an unsatisfactory condition. When this occurred the pipe was broken down and the material taken back and again put through the machine. We found that when this was done the pipe made from this returned material was much less subject to these contraction cracks. This led us to mixing our material as far ahead of the machine as our equipment would allow.

By the time mild weather appeared we had the problem well in hand. During the summer we had occasion to replace our mixer and when we did so we installed a 30-ft. capacity machine in place of the 14 ft. previously used, and increased the storage capacity of our feed pit as well.

We feel that the time element introduced a factor more or less independent of the water content of the mix, that this time element controlled to a certain extent the shrinkage during the early life of the finished product and that this factor was more dependent on time and temperature than on the length of mixing time. The past winter in Detroit has been about as severe as the previous one. Our system of operation aside from an increase in mixing time and the time between mixer and machine is the same as that formerly used. As yet we have had no recurrence of this cracking difficulty and our average pipe strengths with the same mix are running higher than those we previously obtained.

Another indication that the time element might affect the dry mix of the machine compacted pipe operations has come to our attention. We operate a McCracken packer head machine on a portion of our plain pipe production in sizes from 6 in. to 24 in. in diameter. This pipe is of the bell-and-spigot type made in a split jacket, the socket being formed by a rotating packing device of proper shape into which the packer head or barrel-forming device telescopes while the socket is being formed. After the material is packed into the bell end of the form the packer head by means of a rotating and vertical motion, forms the barrel of the pipe.

After compacting, the pipe and jacket are taken to the kiln, placed on the spigot end and the jacket removed. This machine is served by a 30-ft. Blystone mixer using a 1:3 mix with an aggregate having a fineness modulus of from 3.5 to 4.5, dependent on the size of pipe. Unless the machine is carefully adjusted, the kiln floors smooth, the transfer men careful

and the jacket strippers experienced, the finished product in the green state may be subjected to shocks that require the withstanding of minor deformations without rupture.

In striving for that condition of material that would give the greatest ability to withstand normal shock and deformation without rupture, we feel that two facts have been established:

a. That under given conditions of mix, that is, equal aggregate, water, and cement content there is a wide variation in cohesiveness and machineability imparted to the mix by cements from different sources.

b. That these qualities under the conditions stated above may be materially influenced by the time elapsed between the placing of the materials in the mixer and the placement of the concrete in the machine.

We have had cases wherein a mix made with one cement could be readily handled under the conditions described above, while a mix made with another brand of like value so far as our standard tests would indicate, would give us considerable difficulty. We have had instances wherein pipe with a machine slightly out of adjustment would not stand up the first time through the machine. The material would be brought back and again compacted into pipe. Sometimes it would be necessary to put it through a third time. It has been our experience that successive handling of the concrete under these conditions imparts to it an additional cohesiveness each time it is handled.

Whether this condition may be attributed to the fact that successive handlings have the effect of additional mixing or whether it may be due solely to elapsed time we do not know. We are inclined to believe that both elements are of sufficient importance to warrant a much more scientific investigation than we have been able to obtain under practical operating conditions.

Mr. Merriman

THADDEUS MERRIMAN* (*By Letter*).—The water-cement theory is commonly understood as applying broadly to all portland cements and the inference is often drawn that any cements when used with equal quantities of water will produce equal results.

So broad a conclusion can hardly be justified. If all cements were alike, such an inference might be correct. Since, however, it is well known that the characteristics of different cements vary within fairly wide limits, it is hardly to be expected that they will all react alike, even if mixed with the same proportion of water.

The higher grades of cement naturally will produce better results than those of lower quality. All of the evidence now available appears to indicate that the very best result of all is to be obtained by the use of the highest grade cement with a minimum of water.

These considerations must be given attention, if the full measure of benefit is to be had from the application of the water-cement theory.

Mr. Lord.

A. R. LORD.—I might state my general reaction to this entire program of the two sessions. It seems to me that I can agree heartily with what Mr. Earley said, because it appears that even his wet mixtures are not excessively wet, but there is a danger in the drawing of general conclusions from experiences in a particular field. I can see how the abstraction of water, even though it be present in rather large amounts, from a road pavement, by an absorptive base would permit the use of much wetter concrete in that road pavement than is generally used, but I am particularly interested in the reinforced-concrete structure, and I do not see that the experience in sand molds or an absorptive road base will lead to a safe practice if applied to reinforced-concrete building. We have a different problem there which will lead to trouble if we try to generalize from the plain concrete field into the reinforced-concrete field. We have, in the case of the reinforced-concrete structure, a large amount of reinforcement which must be rigidly supported; if you put your concrete in wet and abstract the water, your concrete is going to settle away from the reinforcement and you will lose a great part of your bond. We have any number of instances. There was one structure in Chicago where you could put your hand under the whole mat of reinforcing steel; there was no possibility of a horizontal bond between the level of the reinforcing steel and the mass of the concrete below it. Anyone who knows the least thing about structural design knows that that would be a failure.

I want particularly to discuss the paper of Mr. Bates, in which there are two points with which I disagree. In the first place, you may recall

* Bureau Water Supply, New York.

that I asked about the temperature (as affecting the strength) on the first table he showed us, and he replied that it was the outdoor temperature in Texas at the time of these tests. I know from experience that the temperatures in Texas are subject to violent and wide ranges. If the data are based on Texas outdoor temperatures the matter of the temperature cannot safely be ignored. Would the changes in temperature in the 28 days affect the strength to a considerable degree? If there was an increase in temperature of 20 deg., it might increase the strength by 1,000 lb. per sq. in. It is not safe to draw conclusions from data in which the temperature is not controlled. The temperature factor is equally important with the water-cement ratio.

The second matter on which I disagree with Mr. Bates is on the data he showed in connection with the Lewis Institute tests. He gave us a long table and discussed at some length the last part of it. These tests were made under curing temperatures of seventy degrees for the whole period, running up to five years. They were made on specimens cured in a room full of moisture and spray at all times. I cannot see that such conditions have any applicability to the concrete of an ordinary reinforced-concrete structure. We are much more likely to have a curing period of 70 degrees (or less) for one to five days than for one to five years.

We have a specification of the American Concrete Institute which requires us to keep concrete wet and to keep it at not less than 50 degrees temperature for three days. You can see, considering the effect of temperature and the well-known fact that moisture is essential to increased strength in concrete, that these tests showing that the wet concrete began to gain on the stiffer mixes after a period of six months, have little or no application to the strength of concrete in a building which is dried out at much earlier age by the heating apparatus of the building. This artificial heat is not favorable to increased strength unless moisture is present also.

I also doubt whether we shall be able practically to hold concrete for a period of two to four hours. The test which he showed was held for three hours before the water was abstracted. When he held it fifteen minutes, he got a relatively small increase in the strength. When he held it for three hours and then abstracted the water, he got this large increase on which he bases his paper, as I understand it. I do not believe it takes a seventh son of a seventh son to predict that we shall not hold concrete on a construction job for three hours before we abstract the water. If we abstract it at all, we will do it right away.

Coming now to Prof. Abrams' paper, there is still one matter with which I must express a disagreement; there is no recognition whatever in his re-statement of his water-cement ratio, of the effect of the concrete. I am agreeable to writing the water-cement ratio as Chapter 1 of the book, but I think you will have to write Chapter 2 on the effect on strengths of the aggregate, because that may run up to 50 per cent. On the Wacker Drive we got 50 per cent more strength than Abrams' ratio gives, and we got it consistently for two years, but it was due primarily

to the materials used. In Chapter 3 you have to deal with temperature. If you close the book after reading Chapter 1 on the water-cement ratio, you haven't anywhere near the full story.

Mr. Bates. P. H. BATES.—I agree with Mr. Lord entirely. I think half the time I spend in answering correspondence is used to impress upon people the entire lack of relationship between laboratory and practical tests. I was discussing the water-cement ratio solely on the basis on which it has been presented by others, that is, laboratory tests. In the question that Mr. Lord brings up, of the possibility of extracting excess water by our present method from reinforced concrete, I can only say that again I agree with him entirely. It is not possible to extract the excess water with our present forms, but I am not willing to agree that our present forms are, in the present stage of the concrete industry, the proper conception of what a form should be, any more than from a present viewpoint we would accept concrete practice of five years ago as proper. In the laboratory all of you have filtered more or less material; you have filtered it through a glass funnel; you have placed between the materials you were going to filter and the glass funnel, a medium. Possibly that is the answer in connection with wet reinforced concrete and our present forms—simply the medium to extract the water. And such mediums have been used most successfully.

Construction of the Grandfey Viaduct.†

By Adolf Bühler*

Introduction.—In the United Swiss Rys. there reigns at present a lively activity in bridge construction. The cause for this is electrification, the principal object of which is to make use of the water power of our country, and thereby to make us independent of the surrounding countries. Besides this, the electrification of our lines will enable us to put railway transportation on a new footing and to win back lost traffic through the economical conduct of business, coupled with speed, cleanliness and comfort. Since the beginning of electrification many bridges have been begun or built, but I shall restrict my discussion to a single structure.

RECONSTRUCTION OF THE GRANDFEY VIADUCT

This viaduct is situated on the Lausanne-Berne line not far from the city of Freiburg, Switzerland. It is to be completed by the time of the introduction of electrification, that is, May 15, 1927, together with five other bridges which are to be extensively reconstructed in a permanent manner.

Historical.—The old Grandfey Viaduct (Fig. 1) was constructed in the years 1857 to 1862 by a private railway company. It had a length of 1250 ft. and a maximum height of 270 ft. Until now it was the most notable bridge construction of the National Swiss Rys. It lay in a straight line and had seven spans, of which the five middle ones were 160 ft. long and the two side spans 142 ft. long. At both ends of the iron superstructure there were two abutments, each divided by arches of 16½ ft. clear opening. These abutments lie on the edge of the valley side. The superstructure of the main spans consisted of a latticed truss continuous over all seven spans. It was supported by the two abutments and the six iron piers (Fig. 2). This continuous superstructure consisted of four main trussed girders lying 6.8 ft. apart, and it was calculated for a maximum total load of 2700 lb. per foot of track or approximately Cooper's E 22. The main girders were latticed as shown and were built of wrought iron. In the space between the upper and lower chords of the two middle main girders, there was a footwalk 5 ft. wide, the approach to which was made by two circular stairs within the abutments. The piers consisted of two parts: the lower portions were constructed of carefully fitted large sandstone blocks, while the upper portions were iron towers. These iron tower piers had all the same height of 142 ft. and consisted of eleven stories. Each sandstone base supported twelve iron stanchions of round cross-section braced together to make the iron towers. The 12½ ft. sections of these tubular cast-iron stanchions

* Chief Engineer of Bridges, National Swiss Rys.
† Translated from the German by Prof. George E. Beggs, Princeton University.

were fastened to each other by four strong bolts and were braced against each other by flat-irons. The lowermost cast-iron frame was anchored to the masonry by means of bolts extending 50 ft. into the masonry pier. At the top and bottom of these tower piers were massive cast-iron frames which served as receivers and distributors of load. By reason of the continuity of the bridge both of the outside iron piers had to deflect as much as 1 ½ in. each way for a change of temperature of plus or minus 45 deg. F. In addition to this there was the effect of braking force, which the iron piers had likewise to resist. In an unfavorable case an anchor bolt received 44,000 lb. tension.

The abutments and masonry bases of the piers, whose height according to the level of the ground amounted to as much as 108 ft., were constructed of local

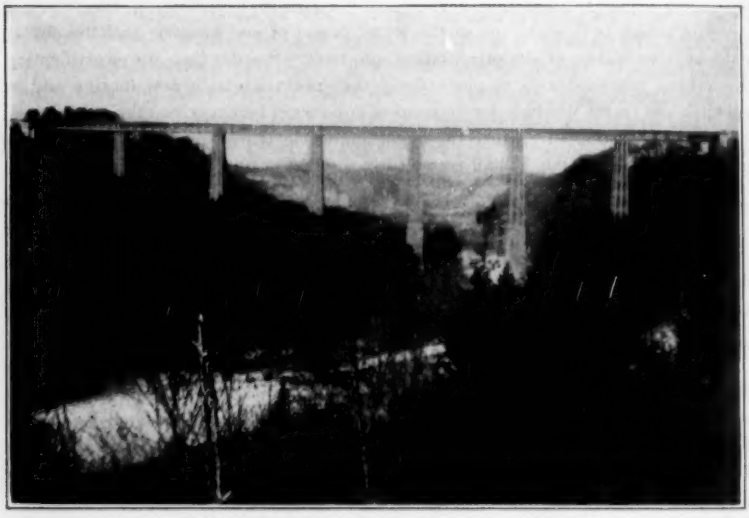

FIG. 1.—OLD GRANDFEY VIADUCT.

sandstone blocks. One stone quarry was situated on the left bank, downstream from the bridge, and another, which, however, was given up shortly after the beginning of the construction, lay upstream on the right bank. In order to economize on material, the foundations of the piers and abutments were provided with hollow spaces, which hollows make up a total content of 3,000 cu. yd. In consideration of the fact that sandstone, wet from the quarry, shrinks considerably in the air, an equal drying out of the sandstone from both sides was accomplished by ventilation through these hollow spaces. A pier foundation took about a year to construct.

In correspondence with the custom of the country, the sandstone masonry was covered over to a level above high water by tufaceous limestone of Corpataux. The edges of the heads of pillars IV and V which stand in the Saane River were protected by good limestone. This viaduct cost, inclusive of some slight alterations

and strengthenings which were carried out in the years 1889 and 1897, about $500,000. In this, a cost of $22,000 which was awarded to the builders by a court of arbitration is included. (The judges of the court of arbitration were Didion, Jullien and Talabot, chief engineers of the then existing three large French railway companies.)

In this construction were used:

26,000 cu. yd. of sandstone	$100,000
3600 tons of iron construction, of which 2200 tons was for the iron tower piers	200,000
1400 tons for the iron superstructure	160,000
as well as for sundries and later strengthening	40,000

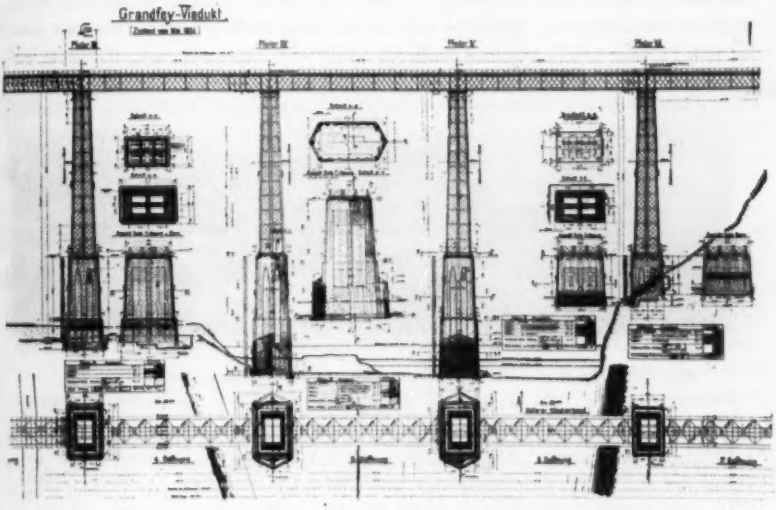

FIG. 2.—PART PLAN AND ELEVATION, OLD GRANDFEY VIADUCT.

The weight of the tower piers consisted of 1440 tons of cast-iron and 770 tons of wrought iron, which constituted about 5000 lb. per ft. of height of the iron piers. The iron construction contained about 500,000 rivets and bolts.

The preliminary plans were made by the Swiss engineer, Durbach, and the two French engineers, Thirion and Jacqmin. The association of the German Chief Council for Construction, Etzel, cannot be proved by any act of his. On the other hand, the engineer Blotnitzki, together with the engineer Nördling, prepared the first plan for the viaduct in 1857 and estimated the cost as $472,000. As prototypes for the construction of the Grandfey Viaduct, the Crumlin Viaduct near Newport (1853) and the viaducts over the Thur, Glatt and Sitter of the Winterthur—St. Gallen (1885) were taken into consideration. The final plans were worked out under the direction of the chief engineer of Usines Schneider et Cie. du Creusot (France). The carrying out of the bridge construction was given over to this firm

on March 3, 1858, as the result of a competition. The substructure was carried out by a Swiss firm Wirth, Studer & Co. and Gribi and Herzog.

The erection of the iron superstructures was accomplished by shifting the trusses forward from the Berne side. This mode of procedure was then entirely new; it was much admired by the technicians and was imitated in many other bridges (Fig. 3). The particulars of this method of erection are the following: First of all the three first spans belonging on the side toward Freiburg were assembled on the Berne side upon a platform constructed in the axis of the bridge and at the height of the lower chord were riveted together. In the meantime, the bringing in of material was accomplished by means of the completed railroad at this end. Then the main trusses of the bridge were set on iron rollers and the structure was moved forward by means of winches until the crane, placed on the projecting end of the bridge, was situated over the completed masonry foundation of the first intermediate pier. With the help of this crane the iron parts of the first tower

FIG. 3.—ERECTION OF OLD STRUCTURE: PHOTOGRAPH MADE IN 1862.

pier were lowered and put in place story by story. At the same time the bridge truss was lengthened by adding a larger span on the ground toward the rear. After the completion of the first tower pier, four rollers were placed upon it, one for each main girder, and on these the superstructure was shifted for the distance of one span until the crane was situated over the next intermediate pier, whereupon the above mentioned work was repeated. In this fashion the six tower piers were erected one after another, in about two months time for each, without special equipment, and at the same time the 1100-ft. lattice bridge was brought into its proper position. During the shifting, the tower piers were bound together at their upper ends by anchorage chains fastened to the abutment on the Berne side in order to produce a sufficient stiffness of the iron towers in the longitudinal direction, so that the piers would not be bent forward as the bridge was shoved across the valley toward Freiburg. Since the main girders projecting over an entire span during the erection had to withstand a significantly greater bending moment than in the completed bridge, the cover plates and the web plates of the chords were reinforced in the neighborhood of the leading span and were further stiffened by a temporary king post truss over the foremost tower piers. The duration of the shifting of a single bridge span length of 160 ft. was about eight to ten hours and this was accom-

plished by sixteen men operating winches. The weight of the part to be shifted was at maximum 1600 tons (including all seven spans with cranes, superstructure, etc.). The greatest speed of shifting was 6½ in. per minute.

The entire Lausanne-Berne line was designed from the beginning as far as the substructure is concerned for two tracks, although at first only one track was laid. Accordingly, in order to save expense, superstructures acting separately were designed for most bridges. With the old Grandfey Viaduct, however, consideration was taken for greater stability, and the superstructure of both tracks was built at the same time and arranged to act together, and the necessary width for the later second track construction of the railroad was provided for. It may be mentioned, by the way, that the piers of the Grandfey Viaduct were originally planned to be built of stone for their entire height. For this 39,000 cu. yd. of masonry would have been necessary. Finally, however, the upper parts of the piers were constructed of cast iron. As reasons for this change, the mediocre quality of the sandstone that was available, the extraordinary height of the viaduct, and the possibility of quicker construction through lessening the volume of masonry, were assigned. One of the proposed alternate schemes of the above named firm, calling for five larger spans (215 ft.) and four intermediate piers built entirely of masonry with hollow spaces therein, was for the same reasons refused by the directors of the railway company. Likewise, a plan for a double cast-iron arch bridge, which would have contained sixteen spans of 70 ft. distance between the piers (plan of E. Martin, Paris) was rejected.

The construction work was begun in August 1858, after the railway company in the preceding year had accomplished various preparatory operations (excavations, etc.). The firm of Schneider & Co. began in February 1859 the casting of the cylindrical columns and the fabrication of the wrought-iron parts in their works in Le Creusot, France. The erection of the iron construction was begun as late as February, 1861, since, as a result of various difficulties, in part of a financial nature, delays occurred.

The acceptance and the first trial loading of the construction took place on Aug. 19, 1862. The loading of a span with four locomotives resulted in a deflection of 0.6 in. The time of construction had up to this time been four years. From the story of this old viaduct the following phenomena are worthy of mention. In the year 1866 a lateral deflection of the bridge and of the pier tops in an upstream direction, to an extent of 1 in. on the Berne side to 1¼ in. on the Freiburg side was observed. This deflection was attributed chiefly to the continued one-sided loading of the piers caused by the position of the original single track which was used only on the upstream side of the bridge. Moreover, it was suspected that the predominating direction of the wind up the valley had assisted. In this deflection the inaccuracy of the construction of the ironwork and in addition the deformation of the pointing mortar may have been of influence. To prevent a further lateral deflection of the piers, the tracks were then laid on the downstream side and the permitted speed of the trains on the bridge was reduced.

In the year 1881, in place of wooden stringers, iron stringers and crossties were put in. At the same time, the track was moved near to a middle position. During this reconstruction, it was determined by a leveling survey that the differences in

the height of the middle transverse girders amounted to 1.2 in. which was rectified by use of metal shims.

As a result of an inspection of the viaduct which took place in 1886, various conditions were objected to. Among other things a whirlpool in the river at pier 5 was noted and recommended to be filled up by a deposit of stone, and the non-observance of the speed limit was pointed out. Particular objection, however, was made to the bad condition of the wooden bearing beams lying between the lattice girders and the cast-iron pier tops, for these were severely rotted, particularly under the outer main girders. Only at the ends of the iron superstructure were expansion roller bearings located. Upon being questioned, the constructor of the bridge, engineer Mathieu, said that he had chosen to use these loose wooden bearing beams in order to procure a certain desirable working together of the pier tops and girders supported by them, and at the same time to allow a not too rigid junction of these parts, which would make the effect of impact dangerous to the cast-iron piers. The wood acted as a cushion. It was debated whether to substitute for the damaged bearing beams, either wedges of impregnated oak or iron supporting blocks, or rocker and roller bearings. In the latter case, the bridge would have had to be raised about 2 ft. in order to attain the necessary height for the bearings. It was finally decided to replace the old wooden bearing beams by new selected oak soaked with creosote. This work was carried out in the year 1889. The water was drained away from the bearing beams to prevent rotting.

In the year 1891 after the Mönchenstein disaster, the iron construction of the viaduct was thoroughly examined. Numerous breaks were discovered in the cast-iron pier frames; these were less numerous in the frames at the pier tops. The struts and the lacing of the tower piers were bent out in many places. Besides this, a large number of loose rivets, misbored rivet holes and cracked irons, were found in the bridge construction. In the year 1895, the task of rectifying this was begun and the struts of the tower pier which had bent out were shortened and so straightened by driving in wedges.

The examination of the viaduct which took place in the years 1896–97 resulted in the discovery that with the exception of the longitudinal girders and of the wind bracing, the bridge conformed to the ordinance of 1892, provided that the superstructure was traversed by only one track laid exactly in the axis of the viaduct. Therefore, in the year 1897 new longitudinal girders were laid, and in the year 1899 a new wind bracing was built at the height of the under chord of the main girders. Making use of the opportunity given by this reconstruction work, an exact inspection was made of the cracks in the cast-iron frames at top and bottom of the iron towers, as well as of the distribution of material in the main girder chords. The 5 ft. open footbridge within the iron superstructure was given a flooring of reinforced-concrete slabs. Moreover, it was ordered that in consideration of the cast-iron piers and the cracks found in them, the viaduct might be crossed by trains at not more than 25 miles an hour without applying brakes. The cracks in the cast-iron piers were examined every month as well as the piers in general. Following this, the viaduct gave no cause for official complaint.

Results of the Examination in 1922.—This examination of the viaduct, necessitated, as has already been mentioned, by the introduction of heavy electric loco-

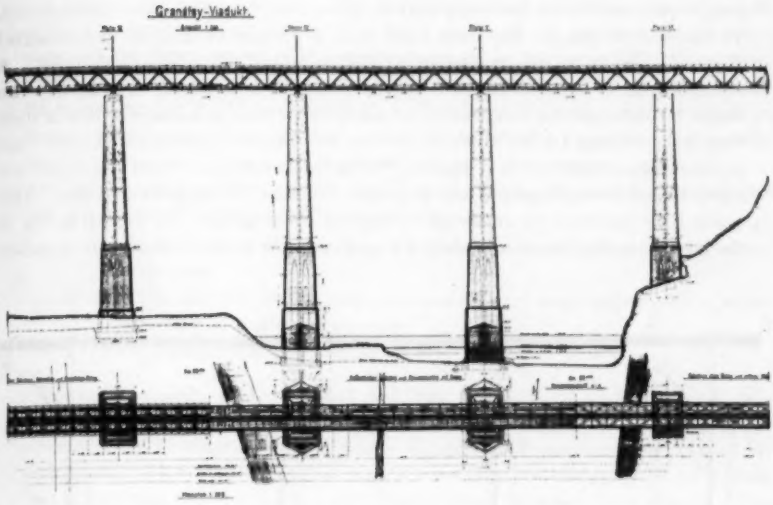

FIG. 4.—DESIGN 1 OF NEW BRIDGE WITH STEEL SUPERSTRUCTURE.

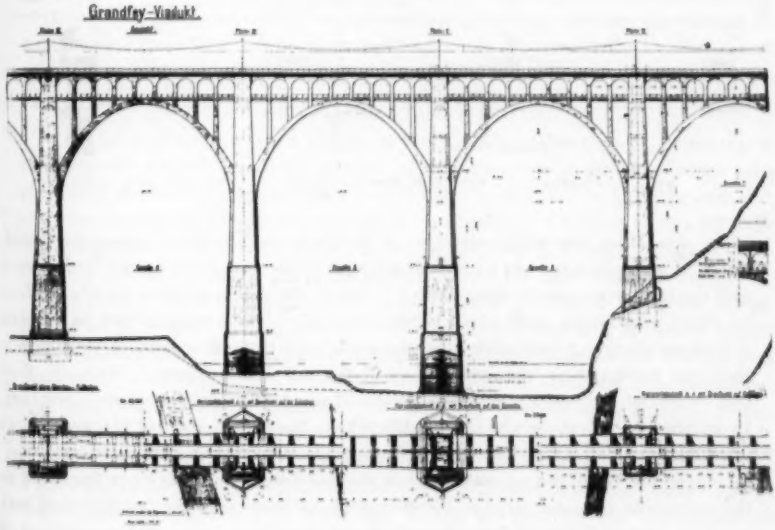

FIG. 5.—DESIGN 2 WITH REINFORCED-CONCRETE.

motives, resulted in the discovery that the piers must be considerably strengthened. The modern weight for one track equivalent to Cooper's engine E 40 represented about the weight for two tracks for which the bridge was originally designed. A strengthening of the iron construction by means of new girders or trusses, which would be sufficient for two tracks, as would have been demanded within a short time by increasing traffic, would have been too expensive and could therefore not be taken into consideration. The only thing that might have been considered was a provisional strengthening of the iron superstructure for single track traffic. This would have included the renewing of the transverse girders, the strengthening of the lateral bracing, the reinforcing of a large number of the compression members

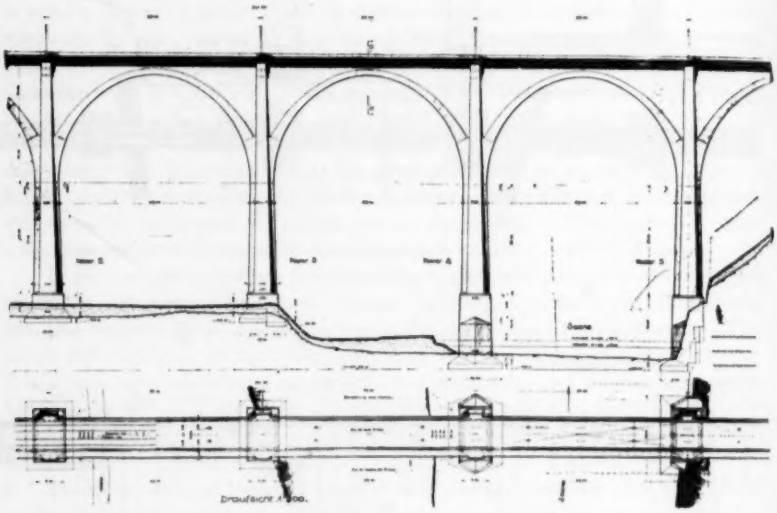

FIG. 6.—DESIGN 3: REINFORCED-CONCRETE BRIDGE ON NEW SITE.

of the main truss, and the construction of girders to resist braking forces, and installation of new bearings. As a result of the rigid connection between the bridge truss and the pier tops, notable longitudinal forces at the pier tops came into operation, as above mentioned, particularly during changes in temperature, and as a result of braking of trains, which made great demands upon the anchor bolts of the masonry piers and produced an unequal loading of the cast iron columns. Since the flat-iron braces of the tower piers were loose and much too weak, oscillation resulted. In addition to this there was the condition that the anchor bolts of the tower piers were inaccessible so that one was in complete uncertainty as to their condition; moreover, one could put no confidence in the cast-iron stanchions when the speed of the trains was increased to 55 or 65 miles per hour because of the brittleness of cast iron and the possibility of flaws in casting and the above mentioned appearance of cracks. Therefore, it was clear from the first that the tower piers must be renewed or embedded in concrete if the substructure of the bridge was to be retained.

Projects for Reconstruction.—The above mentioned provisional strengthening of the iron superstructure inclusive of the embedment in concrete of the tower piers and the improvement of the substructure would have cost $400,000 without producing satisfactory conditions. Furthermore, at the laying of the second track the superstructure would have had to be replaced just the same, which would have demanded once more great expense. Since the building of the second track would soon be demanded, it was decided not to investigate this solution further.

Taking into consideration the condition of affairs, the following schemes for reconstruction were more carefully considered:

1. To set in concrete the existing iron piers and to shift in new steel superstructures.
2. To set in concrete the existing piers and to erect large arches with a superstructure of reinforced concrete.
3. To erect an entirely new arched viaduct.

PLAN 1. (FIG. 4)

The main girders would have once more been built as continuous girders over seven spans. To simplify the building, a separate superstructure was planned for each track. An advantage of this way of arranging the bridge would have been that at first only one new single track railroad would have had to be erected. The cost was estimated at $780,000* for two new single track superstructures and at $566,000 for *one* new single track superstructure to begin with.

PLAN 2. (FIG. 5)

The existing foundation on good sandstone upon which the entire substructure of the ancient bridge was placed would make it possible to convert the viaduct in a relatively simple fashion into an arched bridge as is described below. The cost of execution was estimated at $650,000.

PLAN 3. (FIG. 6)

The axis of this proposed bridge would have laid about 33 ft. downstream from the old viaduct. In consideration of the proximity of the old viaduct only six arches of 160-ft. clear span were provided for in this plan so that the new and old piers would not have been opposite to each other. The cost of this solution was estimated at $1,084,000 for a viaduct of natural stone and $970,000 for a concrete viaduct, including the purchase of land and the new approach track.

In each of these solutions an open footbridge of 5-ft. width across the bridge had to be provided for. In the above named costs, there was, moreover, taken into consideration in plans 1 and 2 the complete rehabilitation of the existing pier foundation, abutments, etc., whose masonry, in spite of sixty-five years standing, was in general, in good condition. The carefully finished outer surfaces of the sandstone blocks contributed materially to this.

From estimated costs it became obvious that of the proposed three solutions only those under plans 1 and 2 could be taken into consideration. Either one of the solutions might have been made the basis of the construction. Meantime, it is to be remarked than an arched viaduct, in comparison with a steel bridge, doubt-

* One dollar was translated as equivalent to five francs. The true costs in America would probably be 2 to 2½ times more than in Switzerland.

less offers great advantages in regard to upkeep. Many railroad administrations in Europe give the monumental method of construction preference, even when it is as much as 25 per cent more expensive than a steel structure. The annual cost of upkeep of the old bridge amounted to about $3000 and this would not be reduced in a new steel bridge according to plan 1, whereas the upkeep cost of an arched concrete viaduct might amount to not more than $600 a year, or one-fifth as much. In large steel bridges, the upkeep of the wooden parts of the structure is an unpleasant matter; in addition to this, burning of ties often occurs as happened on the Grandfey bridge no less than 23 times in the hot, dry summer of the year 1923.

Although, as has been indicated, the condition of affairs was favorable to the execution of plan number 2, nevertheless in order to clear up the financial problem of the construction, solutions by concrete and by structural steel were offered for competition. From the bids which were to be considered acceptable, it became evident that the costs would be:

Solution 1. With one steel single track superstructure..... $530,000
With two steel single track superstructures..... 710,000
Solution 2. A two-track arched reinforced-concrete viaduct..... 612,000

These total costs are somewhat less than the estimates made by the railroad officers; the relation of the sums, however, is about the same.

With regard to capitalizing the upkeep costs, circumstances further favored plan number 2, with the result that the competition gave preference to the double-track arched viaduct built entirely of reinforced concrete.

It was also determined that the width of the open footbridge should be 8 ft. This was just allowable with regard to the execution. A greater widening of this footbridge, which was desired by the people, would have led to difficulties in construction and would have had as the result an avoidable lessening of the lateral stability of the structure.

EXECUTION OF THE RECONSTRUCTION

The execution of this two track arched viaduct was accomplished in the following fashion:

1. *Piers and abutments of the existing bridge* were completely retained. Their cavities were filled with concrete. During this process, steel reinforcement was provided where necessary and cement was injected into the old masonry. In breaking open the old cavities the opportunity was given of making certain of the good quality of the old masonry and the old foundations. The greater number of the joints were completely filled up with mortar. Each pier contained four hollows of 5 ft. x 12 ft. cross-section and of a height varying with the level of the ground to as much as 58 ft. To be able to clean out the refuse of building which was to be found in these hollows, openings were made somewhat above the ground through the partition walls. Moreover, two holes were made on the top of each pier which likewise served as entrances to two adjacent hollows and through these the concrete was let down into the hollows. The inner surfaces of these hollows (5100 sq. yd.) were not smooth, rather the blocks projected as much as 3 ft. so that thereby a good connection with the concrete resulted. Before the introduction of the concrete the inner surfaces of the hollows were carefully cleaned and washed in order to produce a bonding of the concrete with the sandstone, which actually took

place, as was observed. Three months after the finishing of the filling of the hollows with concrete, mortar was injected. In Pier 1, 750 lb. of cement and 500 lb. of sand were injected. This work will be repeated in the near future in order to assure the working together of the concrete and sandstone and the distribution of load between them. The lower solid masonry parts of Piers IV and V were examined as follows. Holes were drilled in them and cement was injected. No great quantity, however, could be injected, and therefore the masonry may be considered sound.

In Piers I to VI, replacement was made of the tufaceous limestone which formed a covering of that part of the sandstone which came in contact with the earth or with the water. The strength of this covering layer was small. Since this stone,

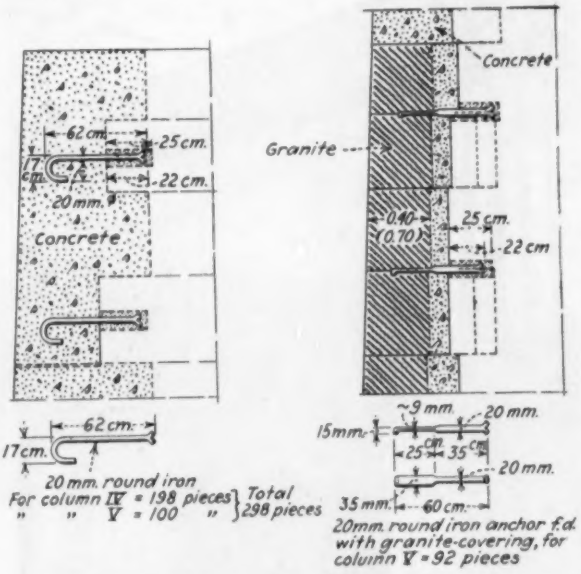

FIG. 7.—TUFF REPLACED BY CONCRETE AT PIER BASES.

though it resists the weather, is not strong under compression, it was decided to replace it with concrete (Fig. 7). The concrete was applied in a plastic condition. In order that it should take part of the load, it was bonded with the masonry. On the outer surfaces of the concrete was put a covering of gunite in two layers. On the outer layer of the gunite the divisions of the blocks of the old sandstone piers were imitated.

Superficially corroded sandstone blocks, of which a few were evident, were chipped down to a depth of from 6 in. to 8 in. and the depression filled up with cement plastered on and trowelled. It was left to a later time to provide the entire sandstone surfaces with a covering of gunite or a similar covering, if necessary.

River Pier V was underpinned in sections after the removal of a deposit of stones inside the cofferdam, where the deepest whirlpool reached as far as the foundation level. This deepening of the foundation amounted to 7 ft.; it was,

moreover, carried out in a circle only around the pier, while the inner center of hard sandstone was left intact. This extension of the foundation received a covering of granite as far as high water surface, in manner similar to the replacement of the limestone facings.

In Pier IV, as well as Pier V, the limestone facing of 30 ft. height was removed in four vertical strips at different times and replaced with concrete to prevent the occurrence of settlement. Moreover, the replacement of the limestone and the

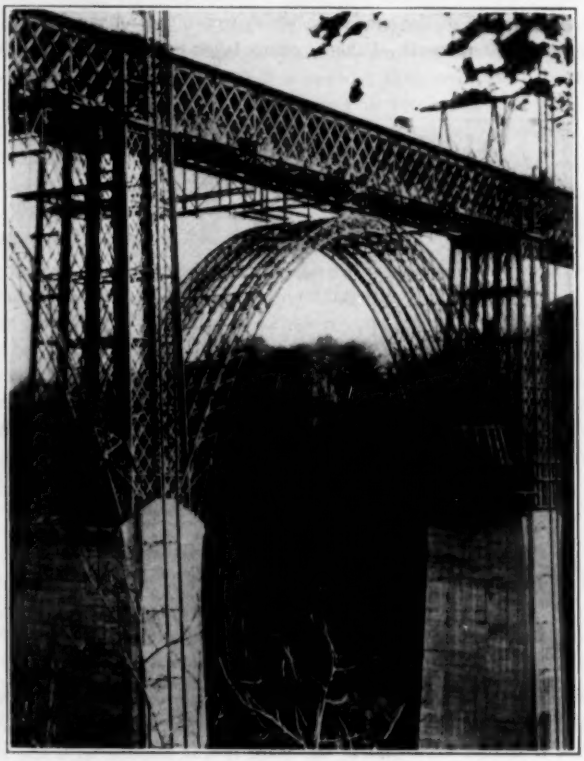

FIG. 8.—STEEL CENTERING FOR LARGE ARCHES.

filling of the pier cavities with concrete was, as far as possible, completed, before beginning the embedment of the tower piers above with concrete, in order to bring the concrete in the cavities under pressure at once.

During this reconstruction of the old piers, the anchorages of the columns of the iron tower piers in the four corners of each pier foundation were examined. The anchor plates were set in cavities and were accessible by breaking through the masonry. It was discovered that a great many of the round anchors and also of the anchor plates were broken. This shows likewise how justifiable the decision was to begin the reconstruction of the viaduct. Most of the broken anchor rods

were on the Berne side, which indicates that a one-sided shearing, as well as temperature and braking power, must have been at work.

2. After the strengthening and improvement of the existing foundations the concreting of the tower piers progressed. This work was accomplished in a relatively simple fashion, inasmuch as the existing tower piers provided an excellent support for the erection of the scaffolding and forms. Access to the places of work was made from above through the open footbridge and down the existing iron ladders in the piers, or from below by ladders from a newly constructed service bridge. It is to be noted that the paint on the old cast iron columns was not removed; it was still in good condition. This paint was thoroughly cleaned and during the pouring of concrete a cement grout was poured along the cast iron columns in order to fill

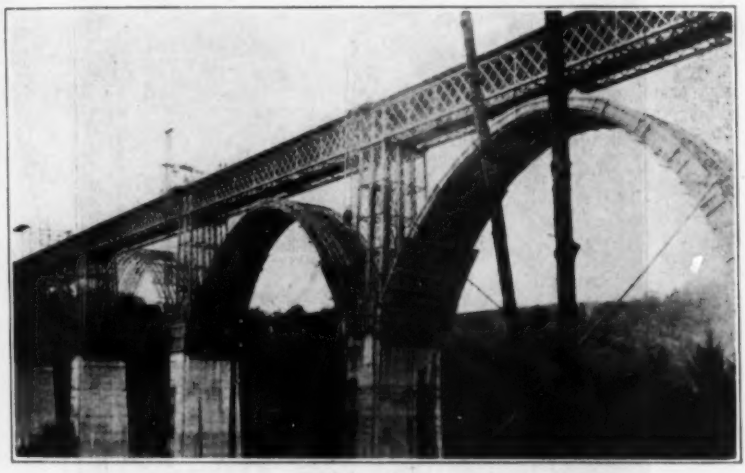

FIG. 9.—ANOTHER VIEW OF STEEL CENTERING.

up narrow openings. Holes were bored in the round columns and these were injected full of cement mortar. In the first operation only about one half of the height of the iron piers was set in concrete. This brought them to the elevation of the springing planes of the great arches.

3. In the erection of the large arches, the predominating consideration was that it would be unavoidably necessary to erect the centering for all seven arches in order to be able to finish the bridge in the specified time of two years. A closer examination of the financial side of this task showed that it would be possible to make use of steel centering without a very great additional expense and that this would serve to support the forms and later serve as the arch reinforcement, according to the Melan system (Fig. 8). In comparison with seven complete, but very light, wood centers, the use of steel centering resulted in an additional expense of only $10,000. In taking over this additional expense, the predominating consideration was that all the material for the centering would remain in the structure in a useful form.

The large arches have a theoretic span of 138 ft. and a rise of 56.5 ft.; the end arches, although of the same form, are about 13 ft. shorter. The body of the arch was put together in five sections. The arches are 3.9 ft. thick at the crown and 7.8 ft. at the abutments, and 27.4 ft. wide above and 30.5 ft. below. The thickness of the arch from crown toward the abutments is reckoned according to the formula

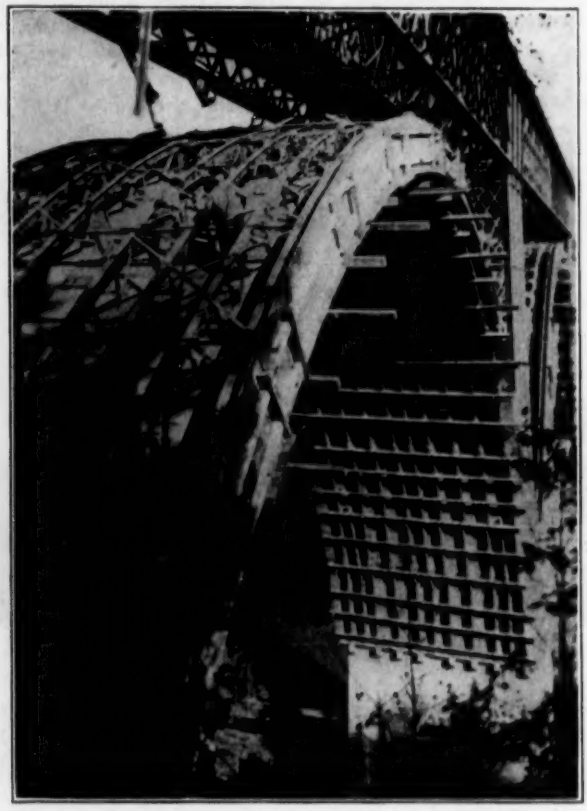

FIG. 10.—CONCRETING THE FIRST RING OF THE ARCHES.

$3.9 + 0.0003\ b^3$, where b equals the distance along the arch in feet (*i.e.*, the increase in thickness varies as square of the distance from the crown).

For each arch eight Melan arch ribs were planned, which were braced together by transverse frames and were bound together by wind bracing of steel flats. The wind bracing was so designed that it could not only withstand a wind pressure of 20.5 lb. per sq. ft. of exposed surface, but also a side component in addition coming from side buckling effect or oscillation equivalent to a force of 3 per cent of the weight of the ring of the arch first poured. The centering was hung to these girders

(Fig. 9). The Melan construction weighed 530 tons in addition to which there was about 58 tons of round reinforcing rods, which were hung around the Melan construction in order to avoid cracks in the concrete from shrinkage and to insure an equal distribution of the pressure of the pillars, which consist of two parts.

The casting of the concrete of the large arches was accomplished as follows (Fig. 10). At first the lower third of the arch depth was poured and its upper surface was provided with transverse ridges 24 in. wide by 4.75 in. high. As soon

FIG. 11.—ERECTING STEEL CENTERING (MELAN SYSTEM).

as the concrete of this first layer had sufficiently set, which as a rule took place in seven days, the second layer or ring, which formed the completion of the arch, was constructed. This construction in layers was desirable in order to keep the cost of the Melan arches within economical limits. The working together of the two layers of concrete is assured through the above-mentioned transverse ridges and through the struts and diagonals of the Melan arches. Each of these constructive provisions in itself is able to carry over the shearing forces which come into play between the two layers. Since the second layer as a result of processes of shrinkage, lays itself firmly on the first layer, a separation of these layers is never to be feared.

The erection of the Melan arches forms an interesting operation (Fig. 11). They were made of steel with a strength of 51,000 to 67,000 lb. per sq. in., a ductility of 25 per cent, and a yield point of 34,000 lb. per sq. in. The bringing up of the arches, each of which consisted of four parts, was done on the Berne side, where the unloading was accomplished with a gantry crane which reached over the railroad track. There, on a platform, the arch sections were riveted together with the transverse frames in pairs and then were loaded by this same crane upon rolling trucks and moved forward on an upstream temporary track out to the proper span. Here was situated a special construction crane under which the traffic trains could pass without difficulty. With the tackle on the cantilevers of this crane, the sections of the arch, about 50 ft. long and weighing as much as 5.6 tons were raised from the trucks, transferred laterally out to clear the old iron bridge and lowered. Under the bridge there were six transverse tracks, which were provided with trolleys. The sections of the arch which had been let down from the cantilever crane were moved back a little under the bridge and then transferred to the chain hoists on the trolleys of the above-mentioned transverse tracks. Thereupon the trolleys carrying the steel centers were drawn back under the bridge and the sections of the arch were let down by means of the chain hoists into the places provided for them and made fast. The old iron bridge never had to carry at one time more than one pair of Melan arches, about 21 tons in weight, which were insured against lateral displacement by two braced arms which were let down from the bridge. In this manner heavy loads hanging on the old bridge were avoided. To be able to erect the Melan arches correctly the anchor bolts on the springing planes of the piers were accurately set with the help of steel templets located by accurate surveying methods. The joints of the Melan arches were completely riveted; on the other hand, the transverse bracing and wind struts were connected by bolts and in part fastened with electric welding in order that the bars should not be weakened by rivet holes.

The Melan arches received a coating of cement milk, to the mixing water of which three to five per cent of potassium bichromate was added. This coating lasted very well. The arch centers had to remain unprotected in the open for three months. During this time no appearance of rust was noticeable on this steel construction. The coating adhered all over. With a coating of cement milk without potassium bichromate, scaling and rust flakes would appear in a short time.

Concreting of the arches was carried out as follows (Fig. 12):

At first the lower ends of the Melan arches at the springing planes were concreted to a length of about 13 ft. in full thickness and the adjacent section of the pier to a height of about 10 ft. was at the same time concreted along with it. After the complete fixing of the ends of the Melan arch was this assured, the rest of the arch was concreted in the above mentioned layer fashion. With each layer, the section 13 ft. long near the springing plane was concreted first, then the section at the crown, which was about 30 ft. long, and then the haunches. Thereupon the work was carried on once more from below for the second layer as far as the sections at the crown where a 4-in. joint was left open between sections (7) and (9), and this was as soon as possible rammed with cement mortar of high quality. This ended the concreting of the arch barrel, which, as will be described later, occurred in the

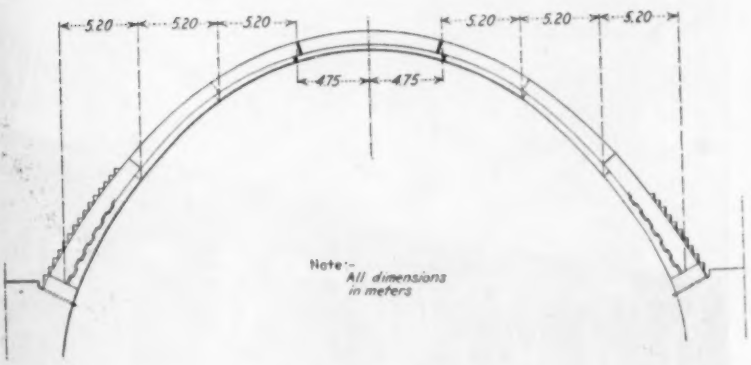

FIG. 12.—CONSTRUCTION PROCEDURE.

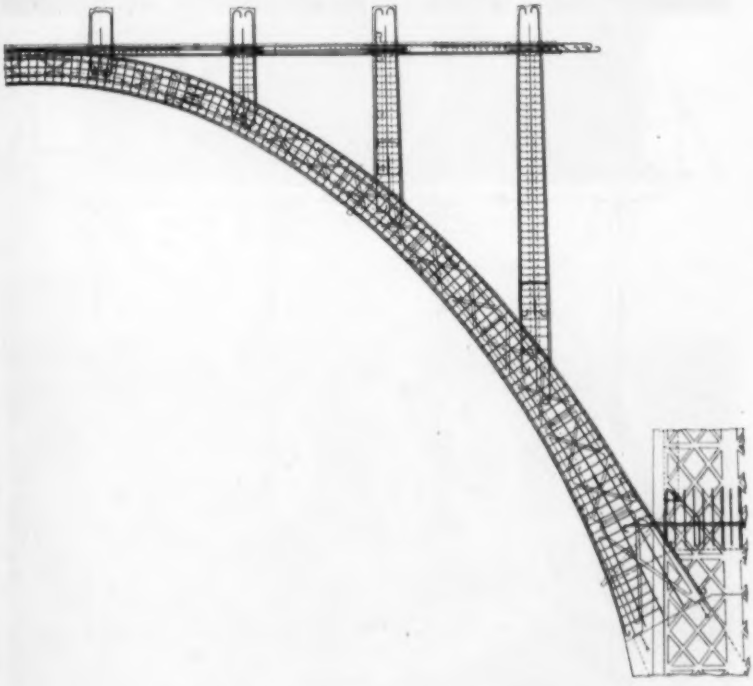

FIG. 13.—SUPERSTRUCTURE REINFORCING.

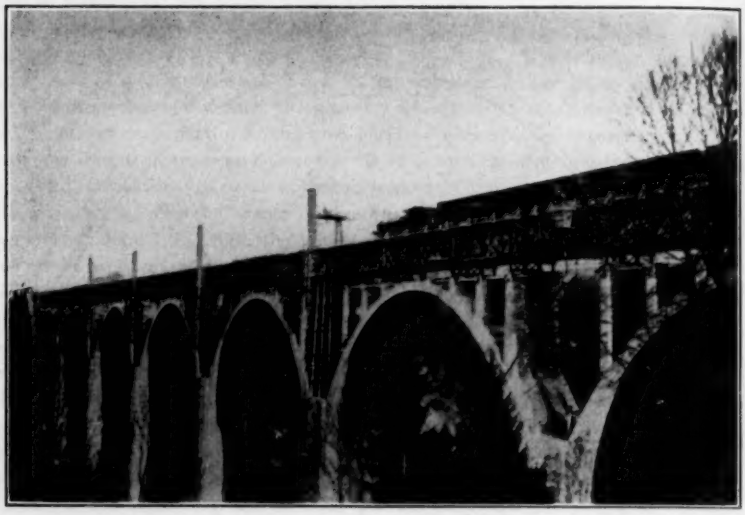

FIG. 14.—STEEL BRIDGE SUPPORTED BY THE ARCHES.

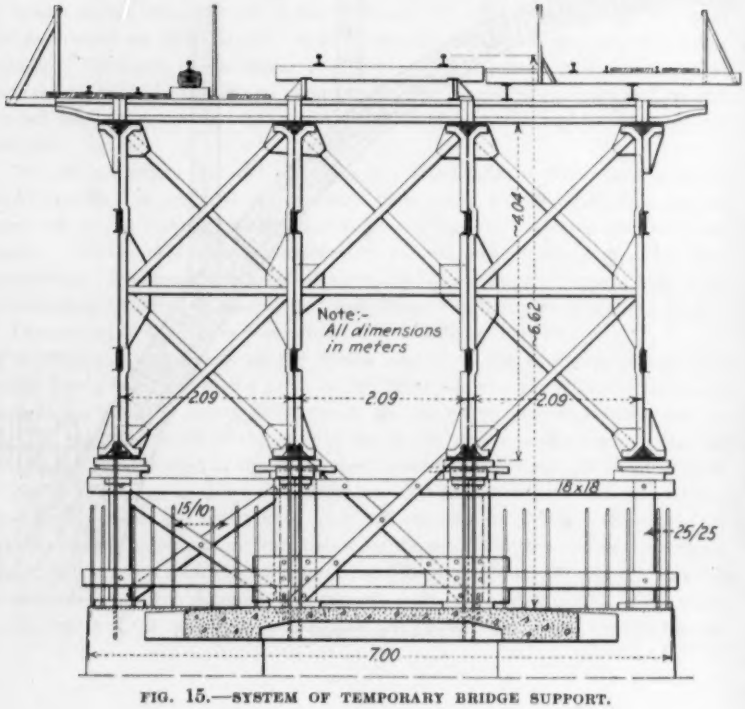

FIG. 15.—SYSTEM OF TEMPORARY BRIDGE SUPPORT.

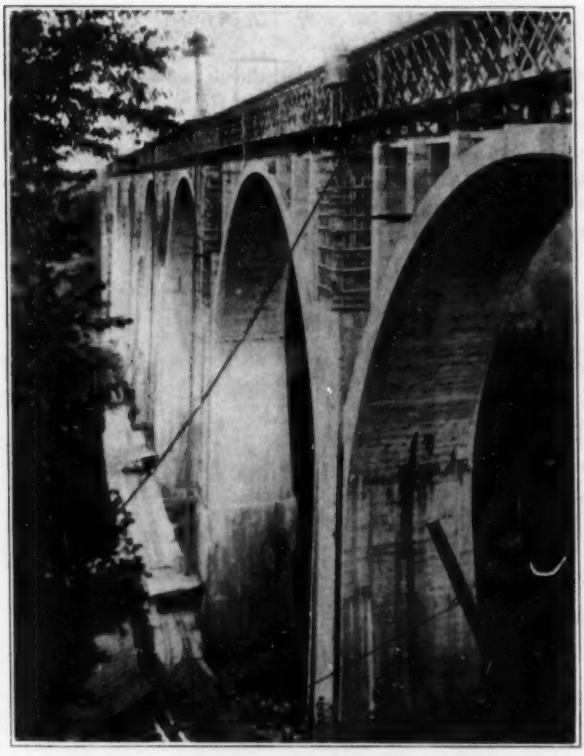

FIG. 16.—TEMPORARY WOOD SUPPORTS.

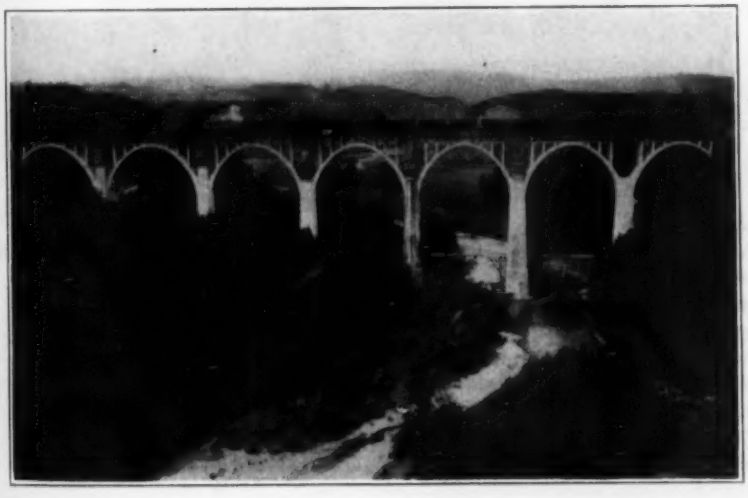

FIG. 17.—STEEL STRUCTURE CARRIED ON CONCRETE ARCH POSTS.

winter of the year 1925. The timber forms, therefore, were left and only removed in March, 1926.

4. The reinforced superstructure was now erected on the arch barrels. This superstructure consisted of vertical reinforced-concrete pillars designed to resist compression and bending, which at the height of the crown of the main arch were bound together by a concrete slab (Fig. 13) and at the very top by an arcade. The superstructure received, for reasons noted below, a very strong reinforcement.

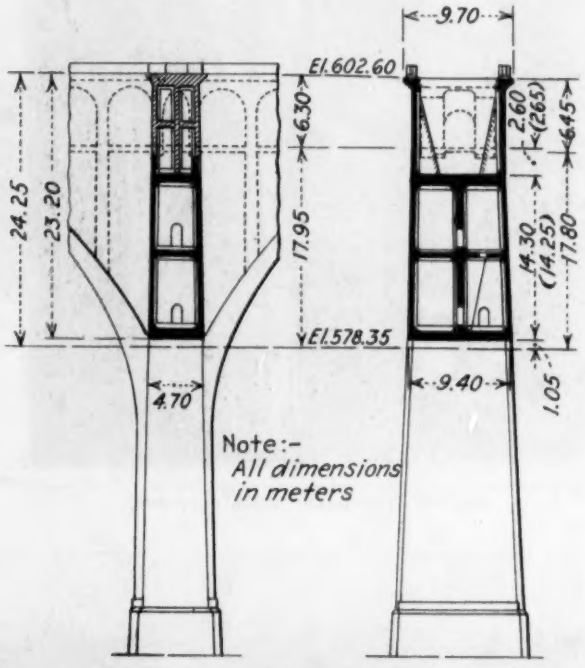

FIG. 18.—UPPER PART MAIN PIER HOLLOW.

It may be mentioned at this place that the large arch barrels, as well as their superstructures, were kept as light as possible in order not to burden too heavily the old pier foundation made of relatively soft sandstone. For this reason, there was at first taken into consideration a still lighter reinforced-concrete construction with *hollow* main piers. Closer study, however, resulted in the discovery that the main piers could be concrete as far as the springing planes of the large arches, as well as the arches themselves, without imposing too heavy a burden on the old pier foundations. This was regarded as advisable in order to produce an appearance of quiet dignity, together with simple erection and low upkeep of the bridge. Nevertheless in relation to the slim superstructure, the principal piers appear very powerful. This, however, produces a completely satisfactory impression.

The superstructures of the arches were next erected up to and including the above mentioned flat slab, which was to serve as the future through footbridge (Fig. 14). This footbridge has a width of 8 ft. between the railings and lies about 6.5 ft. below the level of the ancient footbridge. Nothing more could at the moment be built, since on the one hand the old iron superstructure stood in the way, and on the other the upper parts of the old iron piers. Moreover, the large arches had

FIG. 19.—DEMOLISHING STEEL OF OLD BRIDGE.

to be constructed *under* this iron superstructure, since no part of it could be removed, until a support was furnished. Arrangement was made for this condition by erecting wooden bents over each pillar of the new arcade, upon which the old superstructure could be supported (Fig. 15). This change of support was accomplished in connection with the lifting of the superstructure from the iron piers, which at this time were still intact from the springing planes of the large arches upward. In accomplishing the change of support, the main girders were raised in succession over each pier, the supporting beams of oak on the iron pier heads were taken away,

and then the wedges of the neighboring wooden supports were driven in. By this means the old cast-iron piers were relieved of load and could be demolished down to the heights of the springing planes of the new arches. For this demolition, the simplest means were made use of: loosening of rivets, removal of bolts, and burning off of the iron portions, which were then lowered to the valley floor by means of winches. These old upper parts of the piers would gladly have been kept and cast in the concrete. On esthetic grounds, however, this demolition had to be carried through, for otherwise the upper part of the piers could not be changed to suit the form of the new structure and would present a block like appearance in contrast to the neighboring, slim parts of the construction (Fig. 17). The upper portion of the main piers were now erected hollow in order to avoid weight, which was the more easily possible since the weight of the arcades resting upon them is relatively small (Fig. 18). The hollow construction is carefully protected and stiffened by inner walls. A ladder, laid in the axis of the bridge and leading over the back of the main arch, gives access to the doors of this hollow structure, so that occasional inspections can be made. The hollow structure itself is drained by means of a pipe which leads through the springing section of the large arch and into this drain also the water flowing down over the back of the arch runs. Directly following the erection of the pier tops, the rocker expansion bearings, of which more will be said below, were placed and the footbridge slab was completed.

Coincident with the erection of the tops of the main piers, one of the four main trussed girders of the old iron bridge (the one lying upstream) was demolished and thereby place was made for the first row of arcades (Fig. 19). At the same time the transverse girders of the iron superstructure were cut off in such a way that the reinforcement of the arcade could be placed and yet so that the crane operating on the tracks above could be left in place. These crane tracks were originally laid for the erection of the Melan arches, but later proved themselves very useful for supporting auxiliary crane serving to transport material. Next the pillars of the arcade were concreted as far as the springing planes of the arches of the arcade (Fig. 20). Then came the reinforcing and concreting of the arcade.

As can be seen from Fig. 20, the round reinforcing rods following the under surfaces of these arcade arches are bent in polygonal shape. This was done from the consideration that a loop could be inserted below each angle and this loop could be embedded in the concrete in a more advantageous fashion and with certainty. Moreover, it was assumed that the part of the polygonal under tension, that is the straight bar between the loops, would produce no spalling of the concrete. In the laying of these polygonal bars, however, it was found that it was difficult to bring the angles into line and that it requires the greatest care on the part of those in charge in order actually to realize the unquestionable advantages of bars bent in this fashion. This use of reinforcing, therefore, can only be recommended under favorable circumstances, unless one is willing to run the risk of the advantages being turned into severe disadvantages.

After the erection of the upstream row of arcades had advanced to the fifth main span and the firmness of the concrete could be regarded as sufficient, the web of the upper chord of the nearest remaining iron girder was anchored at every second arcade pillar, both at the upper chord and at half height, and in addition bound with steel cable to the projecting reinforcing bars of the finished arcades.

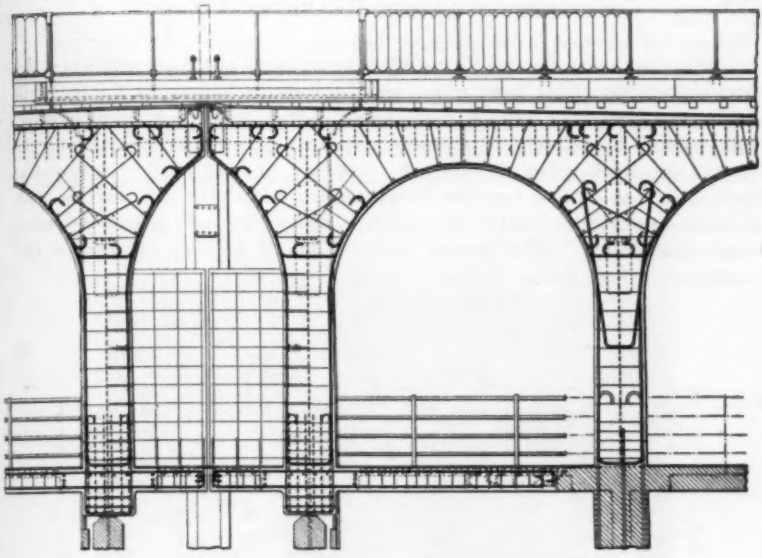

FIG. 20.—REINFORCING SYSTEM IN ARCADES.

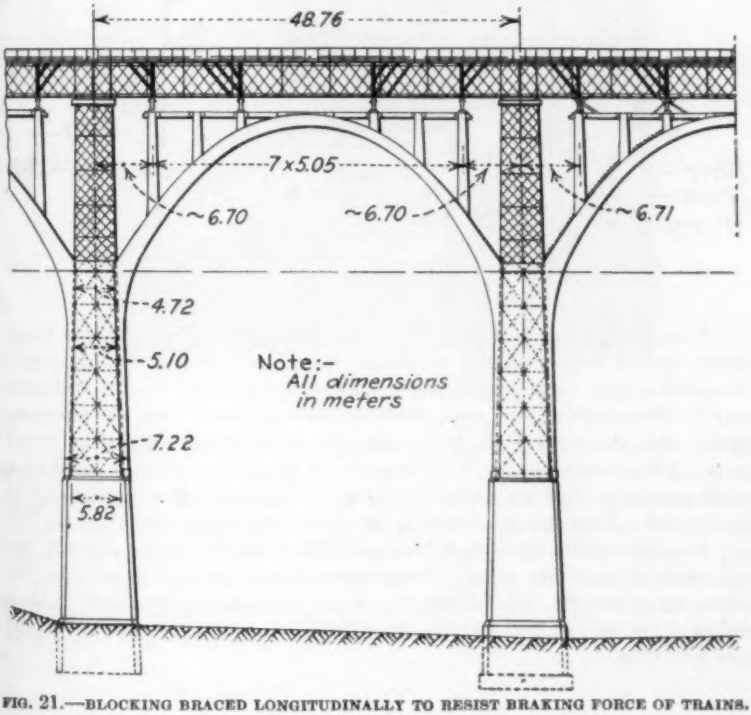

FIG. 21.—BLOCKING BRACED LONGITUDINALLY TO RESIST BRAKING FORCE OF TRAINS.

Before beginning the demolition of the upper parts of the old iron piers, the three main girders lying downstream were strongly held down to the footwalk slab by 10 steel anchors, which were arranged on both sides of the wooden blocking. This was done to maintain the stability in all circumstances. The wooden blocking was also thereby held in line. To resist the braking force of trains, the blocking in span number four was braced longitudinally (Fig. 21), and the iron superstructure at the abutments was in addition insured against excessive longitudinal motion. All anchorages, however, allowed for some expansion of the superstructure from change of temperature.

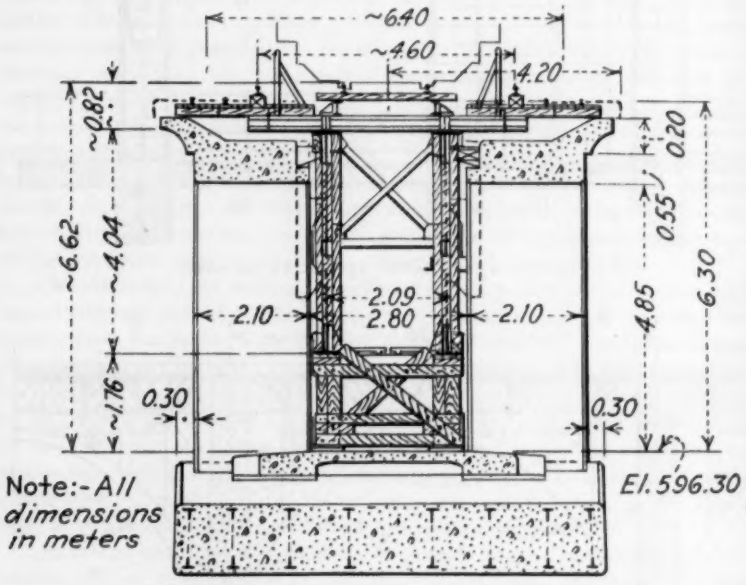

FIG. 22.—SECTION SHOWING TEMPORARY SUPPORT.

Thus both the middle main girders were held in place vertically and transversely at each wooden support and besides this, in each span at six more points, by means of steel cables, which were attached to the upper chord. Moreover, both of these middle girders, whose greatest clear span over a main pier was about 42 ft. and at the crown of the arch about 50 ft., were further stiffened by timber posts. This alteration of the supports of the old iron trussed girder reversed some of the stresses so that some of the flat iron tension members were thrown into compression. These were reinforced by timber to prevent their failure by buckling.

And now nothing stood in the way of demolishing the second girder of the iron bridge lying on the outside downstream side and erecting in its place the second row of arcades. During this time all the trains passed over the two middle girders of the old iron bridge, while the pedestrians continued to pass on the old open footbridge.

Only after both rows of arcades were finished and the concrete had set sufficiently, were the outer ends of the old tranverse girders of the iron bridge, which projected over the arcade, underpinned by wedges. The two middle iron girders were made free from the transverse girders, after bolts had already been substituted for the rivets. Thereupon the traffic was conducted on the track supported by the arcades and the middle main girders were then removed as shown in Fig. 24.

FIG. 23.—CONCRETING ARCADES.

In the work of demolition the method of burning was used almost entirely. For the demolition of the two outer, as well as the two inner girders, a portal crane of special construction was used, which ran on the already mentioned crane track. These girders were seized by the crane, lifted clear of the arches, and let down to the floor of the valley. There, single pieces weighing one or two tons each were brought to the service bridge mentioned below, loaded on small cars, and drawn up on the left bank of the valley on a funicular railroad, and were then transferred to railway cars by a second portal crane.

Those who were in charge of the demolition of the iron bridge and those in charge of the construction of the new viaduct had agreed upon the operation just described. In doing this they were influenced chiefly by the belief that the removal of the iron and the bringing up of the scaffolding, reinforcing, and concrete, would make too great demands on the transportation along the top of the bridge, so that a mutual disorganization of operations would result. This fear was ungrounded,

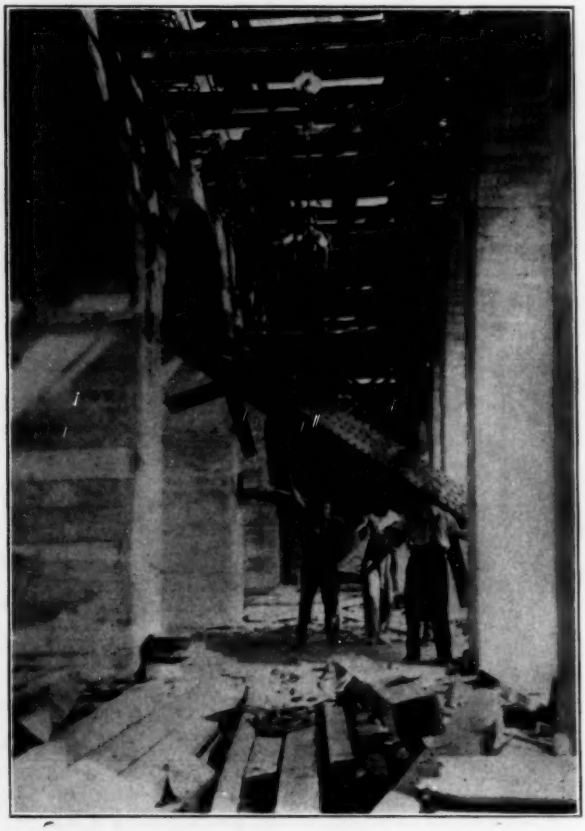

FIG. 24.—DEMOLITION OF INTERIOR GIRDERS.

for the removal of the old iron could have taken place in the natural way over the old iron bridge. On the other hand, the operation chosen had the advantage that those in charge of concrete work could at odd moments busy their crew with the loading of the old iron parts.

After the demolition of the two middle main girders, the placing of the vertical reinforced-concrete transverse ribs, which connect the two rows of arcades, was

begun. The bent reinforcing rods were straightened (Fig. 25), the reinforcing put in place, and then the stiffening transverse ribs were concreted.

At this point the construction was complete in the rough (Figs. 26 and 27), and the traffic continued to pass on the track supported on the arcades upon the old shortened transverse girders and longitudinal girders.

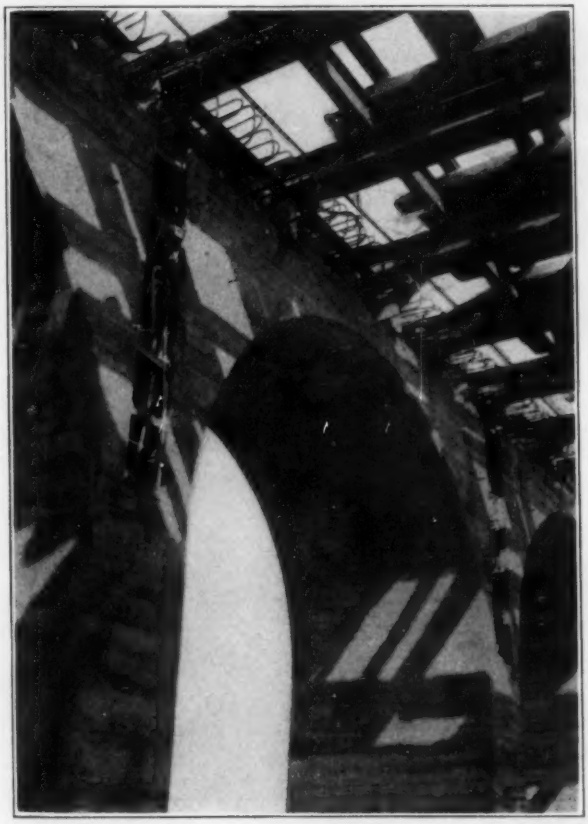

FIG. 25.—READY TO CONCRETE TRANSVERSE STIFFENING RIBS.

5. It was now necessary to place the filling concrete, waterproofing and protecting layers on the rough structure, and to install the expansion joints at the abutments, and over each main pier. In order to obtain a complete waterproofing of the viaduct surface it was decided to carry through an insolating layer (0.4 in. minimum thickness), of clean natural bitumen, with an addition of one intermediate jute. A protective layer of cement mortar 1.2 in. thick was put on the asphalt and wire net was place in this mortar. This work, however, could not be done in its

entire extent at one time because the transverse girders at intervals of 4.5 ft. were in the way. At first they considered leaving these girders, concreting them and simply burning off the projecting parts. The idea was abandoned, however, because then it would not have been possible to carry out the waterproofing layer continuously. Fig. 28 shows how the supports of the track were transferred to

FIG. 26.—ROUGH CONCRETE WORK COMPLETE.

points above the waterproofing in a manner allowing the completion of a continuous waterproofing layer, protected by cement mortar and overlying layers of stone blocks.

The expansion joints over the piers received particular attention in order to prevent the infiltration of water at these generally difficult places. Infiltration would have been particularly unpleasant on account of the footpath lying below (Fig. 29)

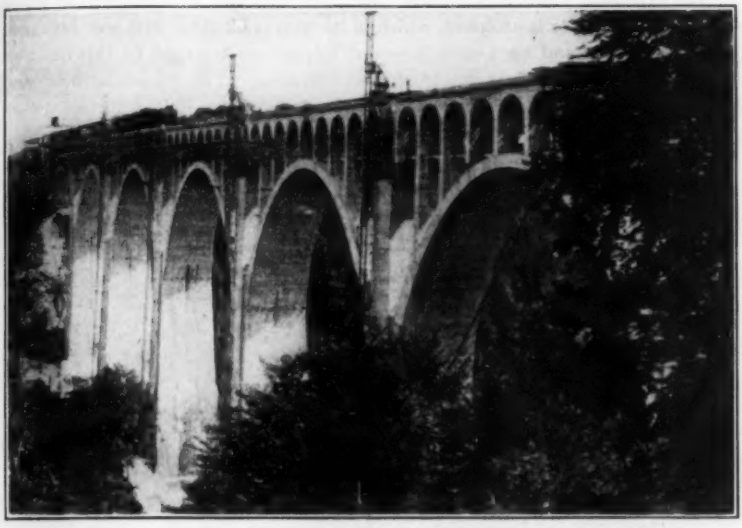

FIG. 27.—ROUGH CONCRETE WORK COMPLETE.

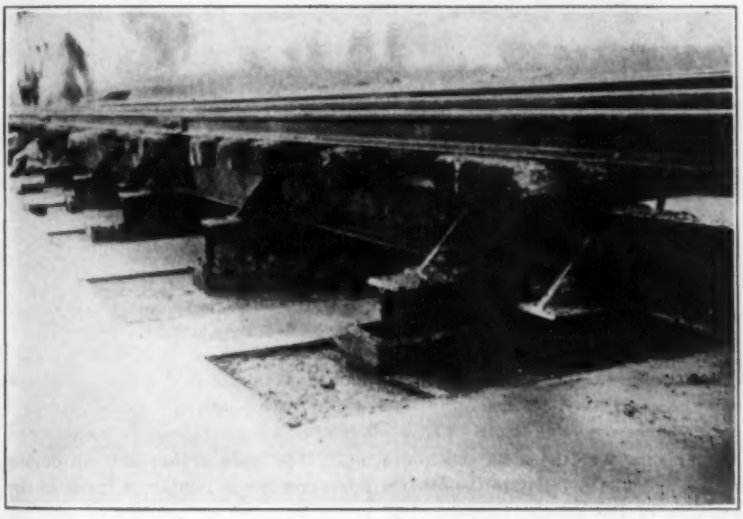

FIG. 28.—TEMPORARY TRACK SUPPORT DURING CONCRETING.

The expansion joints in main floor over piers are edged with flat iron to which a loop of sheet copper is attached, which in its turn is filled up with soft bitumen. The waterproofing and protection layers of mortar are attached to this flat iron and over them a tarred sheet iron is laid for protection. The expansion joints and the crowns of the main arches form the highest places of the drained surfaces, from

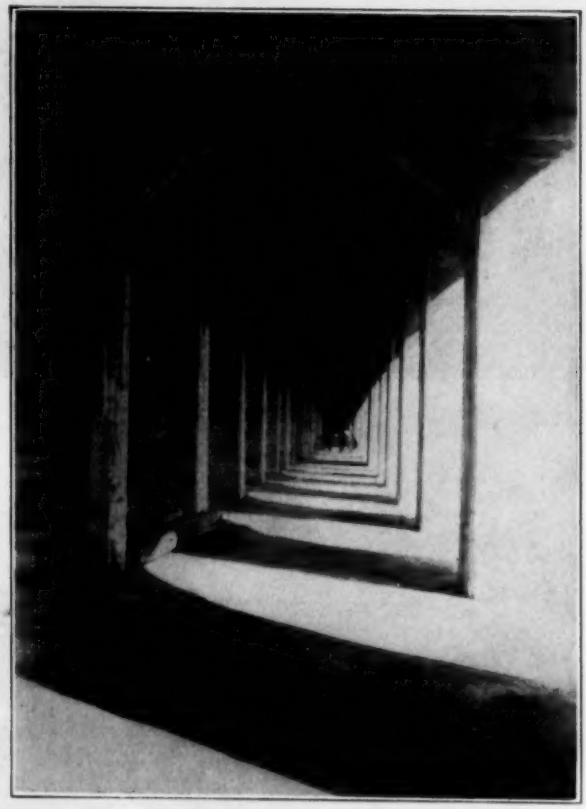

FIG. 29.—PUBLIC WALKWAY OF GRANDFEY VIADUCT.

which the mortar layer falls away to the quarter points of each span. To these points the surface water runs and is led by drainage pipes.

The expansion joints stand in close relation with the constructive building out of the arcades over the piers and abutments. The main arches are continuous from one side of the valley to the other without crevices or joints. A break in the continuity of the arcade, however, had to be included on account of settling, shrinkage, change of temperature, and change of form as a result of the traffic. For this purpose the last arch support of each span at the place of its juncture with

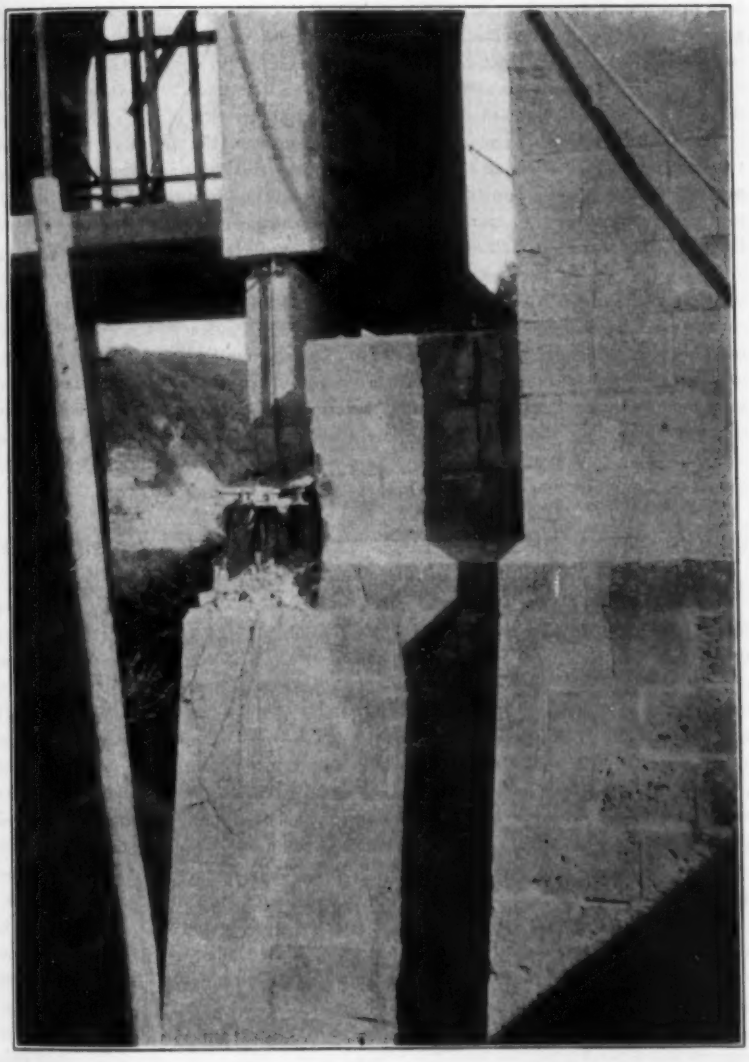

FIG. 30.—ROCKER BEARINGS OF THE ARCADES.

the footbridge slab is mounted on rocker bearings which in their turn rest on the hollow upper part of the main piers (Fig. 30). The arches of the arcade itself project to within a short distance of the middle of the piers, and are there divided by a joint. The last main pillar of each arch and four rocker bearings are concealed in each hollow pier so perfectly that a monolithic pier is presented to the eye, which produces a thoroughly satisfying impression, and gives an advantageous subdivision to the numerous arcade arches (61 in number).

Each rocker bearing consists of a mass of concrete (Fig. 31) 6 ft. high, transversed by eleven flat steel bars 2 in. x ¾ in. Besides this there are ring-shaped loops and other bars. The flat bars are concreted at the one end into the surface of the hollow pier, at the other into the arcade pillar. To prevent excessive bending stress on these bars, there is an asphalt covering arranged around them for 8 in. of length at top and bottom. These flat bars can alone transmit the pressure of the upper structure, while it is assumed, to be sure, that the rolled lead plates at both ends of the rocker can also transmit load. Besides this, at both sides of the rocker, round bars are provided against negative reaction, in case, contrary to expectation, the inaccessible flat steel bars rust or break as a result of repeated bending. The round bars are, moreover, guided above and below by transverse bars bedded in the concrete. On the outside the rockers are protected against the weather by a firm wall; in order, however, to be able to observe the surfaces of the hinges from every side, two removable boards are provided.

PROCUREMENT OF MATERIALS, ARRANGEMENT OF THE PLACE OF CONSTRUCTION AND COSTS OF CONSTRUCTION

The aggregate necessary for the construction was dredged from the bed of the Saane about 650 ft. downstream from the viaduct (Fig. 32). In a working shift of ten hours, about 130 cu. yd. total of gravel and sand could be dredged. The dredge with a theoretic hourly output of 20 cu. yd. brought the material in a dumping dredge bucket to a bin lying about 160 ft. above the Saane on the left bank. From here the gravel and sand was taken in cars to the place of sorting and breaking. This place of preparation consisted of two similar parts, each of which contained a coarse grid, a washing and sorting drum, a stone breaker, as well as lines of transport. The coarse grid sorted out the large stones and carried them directly to the stone crusher, while the material passing through the grid was carried by shaking sieves into the washing and sorting drum where it was divided into sand and coarse gravel for the piers, or into fine gravel for the arches. Insofar as coarser material went out of the drum, it was divided out and put through a stone breaker together with the material coming from the coarse grid. The stone crusher could be adjusted according to the need to produce 1½ or 3-in. gravel and its product divided according to these sizes. Since the material dredged from the Saane contained too little sand, sand had to be manufactured. For this purpose one of the stone crushers has a sand mill. The material produced in this fashion was taken directly to the sand bin at the side of which there was a gravel bin.

For the reinforced-concrete part, three different components of sand and crushed stone were used. The tests of the concrete which were made during the

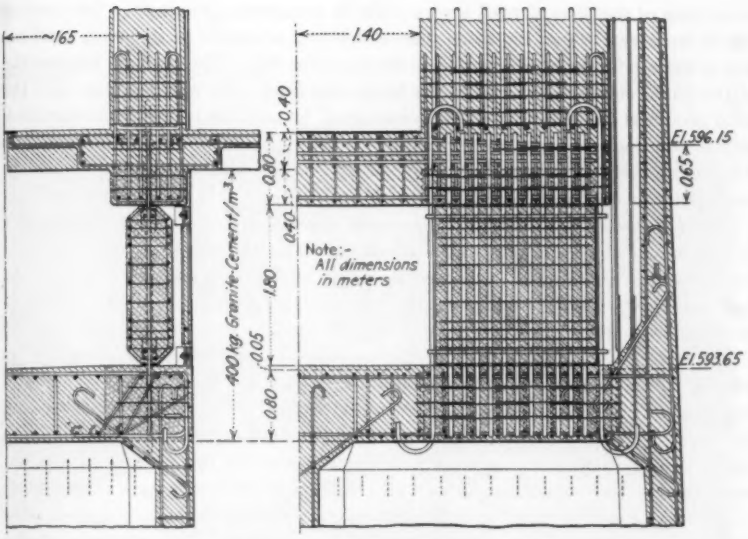

FIG. 31.—DETAILS OF ROCKER BEARING OF ARCADE.

FIG. 32.—CONSTRUCTION SITE.

concreting of the piers showed a very variable compressive strength. For reasons which are given below, the stone dust which was produced in the stone crushers and in the sand mills, was regarded as the cause for this. Therefore, in the making of the reinforced-concrete part of the bridge the sand mills were not used and the sand produced by the stone crushers was freed by washing of the finest particles, as was also done with the crushed stone, in order to remove the dust adhering to the broken surfaces. Since, however, enough round sand was not to be had in the locality of the bridge, it had to be shipped in.

At full speed each concrete mixer could deliver as many as 180 batches in a shift of ten hours. The material was emptied from the concrete mixers at piers 2 to 5 to the elevators which reached up as far as the track at top of bridge. At first one minute was determined as the time for mixing, but later, 1½ to 2 minutes were decided upon.

The supply and repartition of the concrete for the piers was managed as follows. The concrete was elevated and dumped into a hopper erected on the iron pier from which the concrete was transferred by means of pipes to the working places. The amount of mixing water was adjusted so that the concrete was highly plastic. By this elevating and chuting method the cavities of the old masonry piers were first filled up and then the pier shafts above were concreted.

For the concreting of the large arch barrel a distributing plant consisting of pipes and hoppers was used. The highly plastic, often liquid concrete, was hoisted in the elevators, dumped into cars above and run on the iron bridge to the emptying hoppers.

The photographs already shown give an idea of the suspended forms attached to the Melan arches. The bolts with which the forms were secured had to be cut off 1 in. beneath the surface of the concrete after the forms had been removed. For this purpose little blocks of wood were laid around the bolts. These could later be pried out, whereupon the bolts were burned off and the openings closed with mortar. In all, about 4000 such bolts were burned off.

The concreting of the superstructures of the arches was accomplished in a simple fashion. The concrete which was brought up in the elevators and which was mostly highly plastic or liquid, was distributed by means of cars, at some places dumped directly and at other places through pipes into the forms. Particular attention was devoted to construction joints of the concrete. The adjacent surfaces were roughened, cleaned, wetted, and provided with a layer of mortar or fine concrete before the new concrete was laid. Great stress was laid upon the correct placing of the reinforcement and after a little experience a special inspection was instituted for this purpose. To preserve the distance of the reinforcing from the forms, small mortar blocks inclosing binding wire were used.

Finally, it must be added, that for all the forms dressed lumber and boards were used. This measure justifies itself since wood of this kind can much better be used again than unplaned wood. For the careful concreting of the surfaces, a bonus of 20 cents per sq. yd. was paid; about 35,000 sq. yd. were treated in this way. The forms were covered with linseed oil in order to prevent the concrete from adhering.

The entire volume of the concrete amounts to 34,000 cu. yd. of which 17,700 cu. yd. is reinforced concrete. This probably represents one of the most extensive

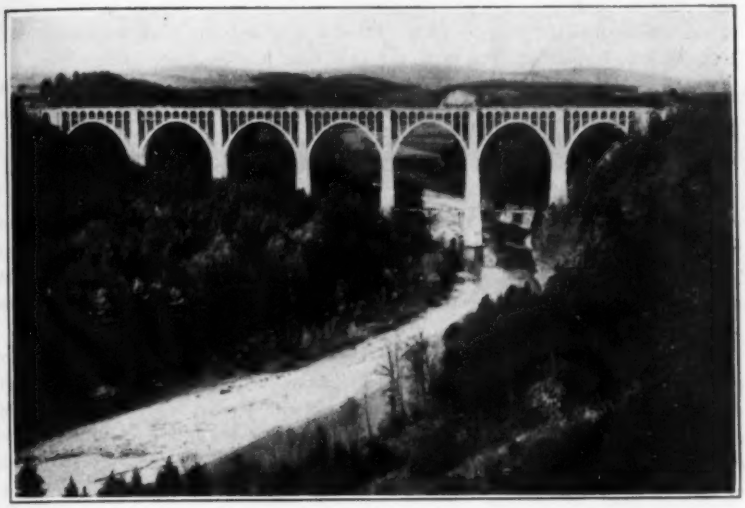

FIG. 33.—PHOTOGRAPH TAKEN IN NOVEMBER, 1926.

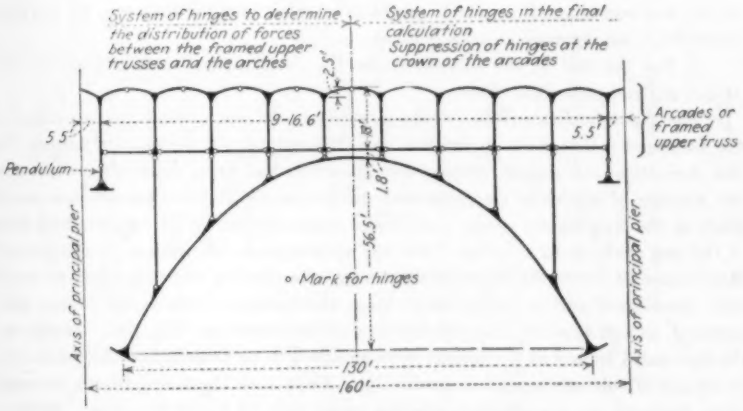

FIG. 34.—HINGE SYSTEM OF ARCADES.

uses of reinforced concrete in the construction of railway bridges. Finally, 1500 tons of steel in the Melan arches and the arcades were concreted. About 33,000 cu. yd. of stone and sand were dredged out of the Saane. A total of 6500 cu. yd. of sand were brought from outside; 8500 tons of cement were used and about 1300 cu. yd. of wood (425,000 ft. b. m.) were used for forms, scaffolding, etc.

About 1150 tons of iron from the old viaduct were concreted in so that, with the new reinforcing, in all 2700 tons of steel and iron are embedded in the bridge.

STRESS CALCULATIONS

Each span of the viaduct forms a combined system consisting of a series of a kind of four-sided, frames of 18 ft. height, formed by the upper part of the arcade with openings of about 14½ ft. and the slabs and beams of the open footbridge. These frames have curved axes; they are attached by means of the lower posts to the large arches, the theoretic spans of the latter being 138 ft. and the rises 56½ ft.

The calculation of the viaduct took place as follows: At first the calculation of the main arches and the piers was carried out with the assumption of rigid abutments and without regard to the stiffening caused by the superstructures. This calculation is final for the effect of the dead weight of the arch barrel, because the stiffening effect begins only after the completion and the setting of the concrete of the superstructure. In the calculation of the arches, the layer method of construction was taken into consideration. And accordingly the dead load stresses in the *arches* were computed in three parts, namely:

1. Stress from the concreting of the first third of the arch in which the Melan arches are alone able to take stress.

2. After the hardening of this first layer of concrete, the second layer of the arches was constructed, in which the Melan arches and the first ring of the arch barrel could be regarded as taking stress.

3. For the rest of the superstructure the entire arch barrel inclusive of the Melan girders, came into effect.

The theoretical calculation of the superstructure was carried out by means of the assumption that each main arch was stiffened by a superimposed frame. In this theoretical calculation, certain simplifications had to be made since otherwise the number of statically undetermined quantities would have become too great. Each of the five middle spans is statically undetermined in 74 respects and each of the end spans in 62 respects. For the approximate calculations of the general distribution of forces the four-sided frames of the arcades were regarded as statically determined and in the apexes of these small arches of the arcade hinges were assumed, as well as at the joints at the level of the footbridge (Fig. 34). Moreover, the four-sided frames of the arcade were assumed to be attached to the main arch by means of columns hinged at both ends. Even with these simplifying assumptions, however, an approximate solution could only be found by steps. First of all the springing planes of the main arches were regarded as rigid and the distribution of load between the superstructure (articulated truss without diagonals) and main arch was calculated. This operation was statically undetermined in eight respects, corresponding to the eight columns which are assumed to be rigid

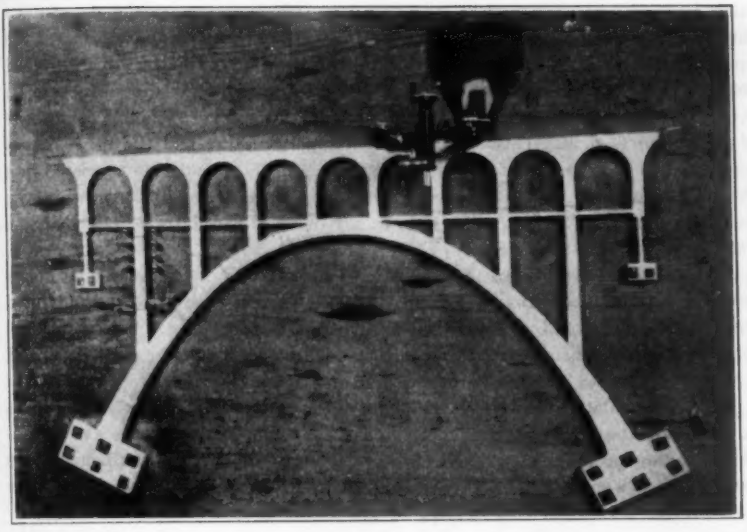

FIG. 35.—CELLULOID MODEL OF BEGGS DEFORMETER.

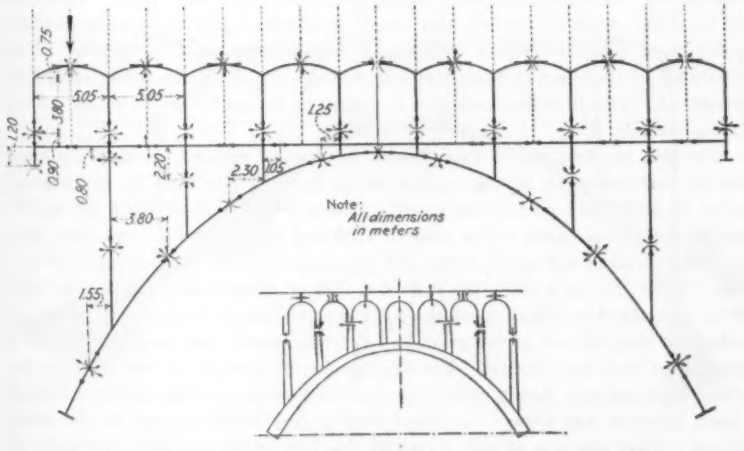

FIG. 36.—TEST OF STATICAL EQUILIBRIUM OF FORCES AND MOMENTS.

and hinged at both ends. These columns connected the three-fold undetermined main arch with the framed-like superstructure, which was statically determined through the assumption of many hinges. It was assumed that the settling of the arch and of the frames through the insertion of these hinges would not greatly alter the general repartition of the forces. In the calculation of the superstructure on the other hand, the hinges in the apexes of the arcades were eliminated. The ratio of rigidity or flexibility between the framed superstructure and the main arches was found to be about one to six. A further step in the calculation consisted in a theoretical consideration of the elasticity of the piers. Space does not allow a detailed explanation of this, other than to say that the method was related to the "ellipse of elasticity method."

The calculation of the arcades was carried out as follows. At first a continuous portal frame with nine spans was assumed. Its columns at their bases were assumed free to turn and bound by a tierod (the footpath slabs and beams). The additional forces in the framed superstructure could be deduced from the already determined repartition of forces between the frames and the continuous, stiffened arches.

As the calculation showed, the influence of the deformation of the arches results in a considerably greater loading of the arcades than that resulting alone from *direct* loading of the arcades. In this, the deformation of the principal piers contributes to a great extent. Since this deformation is much influenced by frictional forces and time, the yielding of concrete will later reach its full effect. Therefore greater stresses are conceded for the superstructure than those for the main arches.

To check to some degree the results of this purely theoretical calculation based on simplifying assumptions, use was made of the undoubtedly unobjectionable "mechanical" method of Professor Beggs, Princeton University, N. J. As can be seen from the above exposition, very many assumptions had to be made in order to carry out the calculations without an exorbitant number of figures. The importance of the effect of these simplifications could not in many particulars be determined. This is particularly true in regard to the pillars, all of which receive bending stresses.

For this our first use of Prof. Beggs' mechanical method, we manufactured with his permission the laboratory equipment in Switzerland, as the preparation had to be made with all possible speed. It was determined to make the experiment on a celluloid model to the scale of 1:50 and at first for only one span (Fig. 35). The shearing and direct forces and the moments were determined in 23 sections. With this as a basis the influence line of these force components were drawn. In the determination of the force components it became clear that the mechanical manufacture of the gages had not been made with as much care as is desirable for such experiments. The displacements produced by our gages at the sections were not pure thrust, shear, and rotative motions, but the principal motion in each instance was always combined with a very small amount of the other motions. That this was so was shown by the test of the statical equilibrium of the forces and moments affecting the separate elements of the structure. In the course of the experiment these forces and moments were corrected on the basis of the conditions of equilibrium (Fig. 36). Thereupon for every position of load

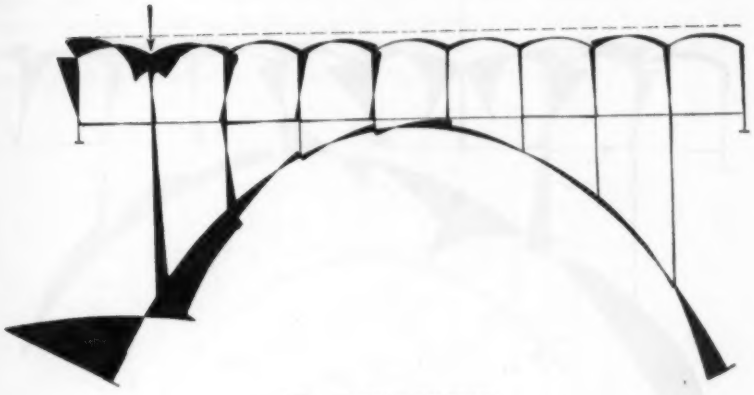

FIG. 37.—MOMENT DIAGRAM.

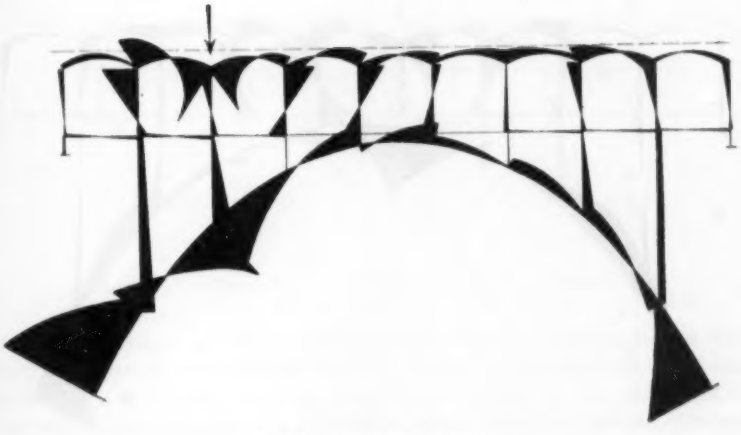

FIG. 38.—MOMENT DIAGRAM.

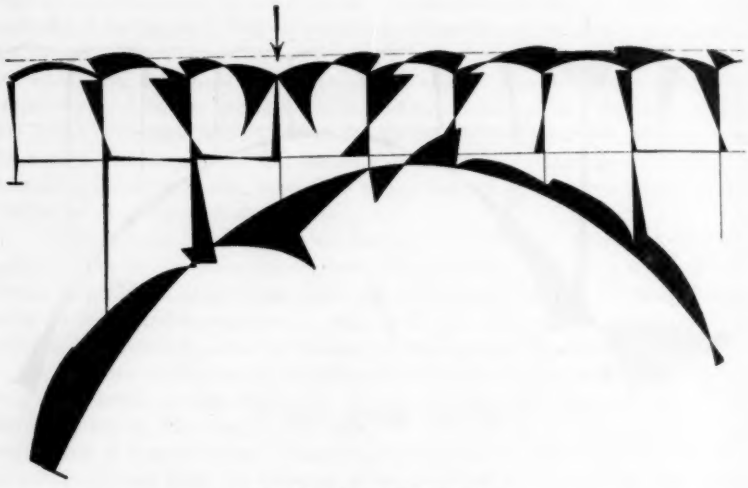

FIG. 39.—MOMENT DIAGRAM.

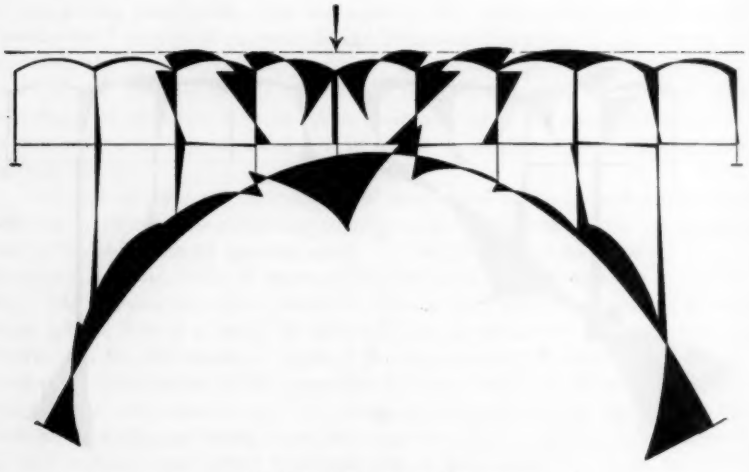

FIG. 40.—MOMENT DIAGRAM.

a corresponding moment diagram of the whole elastic system could be drawn (Figs. 37, 38, 39 and 40). From the accompanying illustration is to be seen in how high a degree the superstructure worked together with the main arches. In like manner starting from the balanced sectional forces, the influence lines for the corner and kern moments were drawn, and from them the boundary values of the forces and moments could be calculated for the actual loading.

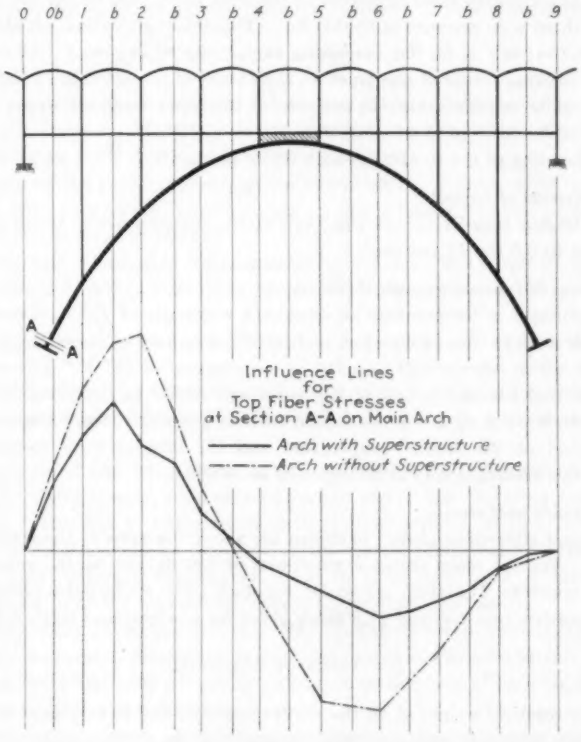
0 0b 1 b 2 b 3 b 4 b 5 b 6 b 7 b 8 b 9
A
A
Influence Lines
for
Top Fiber Stresses
at Section A-A on Main Arch
Arch with Superstructure
Arch without Superstructure

FIG. 41.—INFLUENCE LINES FOR TOP FIBER STRESSES.

The comparison between the theoretical calculation for the simple arch with fixed ends without superstructure and the mechanical results of the deformeter method for a fixed-ended arch with *superstructure* shows in how high a degree the results differ (Fig. 41). A comparison of the theoretical and experimental results for the continuous multiple-arch system cannot yet be given, since the observations on the complete model of the entire structure have not yet been carried out. This comparison will later be made in connection with the working out of plans concerning the loading tests of the actual bridge, which will take place in the course of the year.

A. SUMMARY OF QUANTITIES USED IN THE CALCULATION OF STRESSES

I. *Dead load unit weights:*

Reinforced concrete, 156 lb. per cu. ft.

Rammed concrete, 137 lb. per cu. ft.

Ballast, 131 lb. per cu. ft.

II. *Traffic load:*

(1) A locomotive train ordinance of the year 1919, 7400 lb. per lin. ft. for *each* track, with an axle pressure of 56,000 lb. (This loading corresponds about to the American Cooper's E 60 but consisting exclusively of engines.) Allowance was made for braking forces of one track to the extent of one-seventh of the weight of the load under consideration; in the case of the other track allowance was made for the tractive force of two locomotives.

(2) Loading of the footbridge with 80 lb. per sq. ft.

III. *Coefficients of impact:*

For loaded lengths (L) of less than 50 ft., allowance was made for impact equivalent to 0.6 (50-L) per cent.

IV. *Changes of temperature and shrinkage.*

For changes in temperature an allowance was made of 27° F. above or below the temperature at time of erection, and for the shrinkage of concrete an allowance was made which corresponds to a fall in temperature of 18° F. The assumption was made that about one-half of the shrinkage would be compensated for by a temporarily leaving open joints adapted to that purpose. Since the main arches were erected at the lowest temperature and the arcades were erected in cool weather, this assumption is to be regarded as fulfilled.

V. *Permissible unit stresses:*

The unit stresses employed in design are about the same as those employed in America. For the main arches a maximum of 590 lb. per sq. in. compression is given for concrete, 11,400 lb. per sq. in. for steel. For the arcades forming a part of the superstructure 750 and 17,000 are given for concrete and steel, respectively.

B. TESTS OF MATERIAL

1. *The Steel:*

The theoretical weight of all the reinforcing that had to be placed in the main arches, piers, and arch superstructures (exclusive of the old foundations) amounted to 930 tons. This material was tested by means of 187 series of examinations, consisting each of a test for tension as well as tests for bending in heat and cold; therefore a series of tests was made on each 10 tons.

The collective average of 187 tests of reinforced steel gave as results:

Elastic limit, lb. per sq. in.	42,600
Tensile strength, lb. per sq. in.	57,000
Elongation, measured on a length of 8 in.	31.4 per cent
Coefficient of quality, lb. per sq. in., which means the product of tensile strength and elongation	18,000

Even though the strength of the steel is not particularly high, the limit of elasticity can in compensation be called fairly high. Since the limit of elasticity

of the reinforcing steel is the leading consideration for the load-bearing capacity of a reinforced-concrete structure, it may be considered a fortunate circumstance that, despite the low strength under tension, the limit of elasticity lies relatively high. The high percentage of elongation is desirable on account of the bending of hooks. All the hooks were bent with a clear radius of 3.5 times the diameter of the round irons. The strong reinforcement of the footbridge beams alone received hooks with a clear radius of 2.5 times the diameter. This was done because the cross-section of the concrete did not offer much space. These smaller hooks were bent hot.

2. *The Cement:*

From every 50 tons of cement a sample was taken and examined at the Swiss Testing Laboratory for Materials for its essential characteristics. On the basis of about 100 tests the following results were obtained:

Strength of the normal mortar under compression: 7 days, 5700 to 8500 lb. per sq. in., with an average of 7100 lb. per sq. in.; 28 days, 7100 to 10,000 lb. per sq. in., with an average of 8500 lb.

Strength under tension of the normal mortar: 7 days, 460 to 600 lb. per sq. in., with an average of 500 lb.; 28 days, 570 to 680 lb. per sq. in., with an average of 640 lb.

Rise in temperature of the mortar 2 deg. F. to 11 deg. F. with an average of 7 deg.

Beginning of setting: three to five hours with an average of four hours.

Time of setting (calculated for 60 deg.), 8 to 12 hours, with an average of 9½ hours.

The results of the tests are relatively constant. The strength under compression and under tension were extraordinarily good; also the sieve test showed good value. The rise in temperature showed small value. It is to be doubted whether our usual method of determining the rise in temperature of the mortar is of any great practical significance, since in large quantities of concrete the absolute quantity of the released warmth is the determining factor.

The beginning of setting and the time of setting are of importance for the constructing engineer. According to Swiss standards in the case of slow setting a period of at least one hour for the beginning of setting and of at least two hours for the time of setting is prescribed.

I have made a point of obtaining a decidedly slow setting cement. The minimum delay before setting began amounted to three hours and the time of setting to eight hours, calculated for 59 deg. F. A long delay before the commencement of setting is desirable, for in this time the mixing, the transport, the occasional remixing as a result of separation during transportation, the placing in the forms, and the rodding and working of the concrete must be attended to.

In concrete there is the additional consideration that the workmen have to stand a good deal on the concrete in order to work it, and also the consideration that the introduction of the concrete gives rise to deformation and jarrings of the scaffolding, the bad effect of which often shows itself only too clearly through the formation of cracks. In the case of the construction joints in the concrete, which in themselves represent a poor measure, the above-mentioned unfavorable

circumstances can only be met in an incomplete fashion. The ideal circumstance, therefore, would be the longest possible delay before the beginning of setting, that is, at least eight to nine hours, corresponding to the period of work. During the night in the rest interval, the setting could take place; so that the concrete, which had been brought in, would have set, when the work was taken up again. Naturally the periods specified by rule are altered through the conditions of the weather: particularly by the temperature; the periods are shortened by a rise in the temperature and lengthened by a fall. The laws controlling this in concrete construction have not been sufficiently investigated.

As has already been mentioned, coarse material was predominant in the material dredged, combined with a very fine sand with a large content of quartz (50 per cent) coming from the corrosion of sandstones, over which the Saane flows for the greatest part of its course. It was clear, therefore, from the beginning that only a sorting of the material would give satisfactory results. Therefore, a sorting of the sand with a grain of zero to ⅛ in. and of the gravel with a grain of over ⅛ in. was carried out. Since, however, the sand was lacking to a greater degree than had been anticipated, it had soon to be manufactured to a great extent by means of sand mills. Before the actual work of concreting was begun, a granulometric examination of the sand and gravel material was carried out. Moreover, compression tests of cubes of concrete were undertaken.

The strength after three months was determined by means of cubes taken from the structure. A large number of cubes are still at hand for the purpose of determining the strength after a long period. The preliminary results were very good, particularly in consideration of the fact that very plastic concrete had been used. Then, however, the strength fell off sharply so that the values of the 7 and 28-day tests often could hardly be distinguished from each other. This circumstance resulted in an investigation of the causes. It was soon recognized that in addition to the insufficiently washed dredged material, the stone meal produced in the breaking of the stone material and in the sand mills must, before everything else, exercise a dangerous influence. On the one hand the stone meal clung in a fine coating around the wet stones and prevented a complete adhesion of the cement mortar, and on the other hand, it seems not impossible that accidental pockets of the stone meal might be at work. In particular, the sand produced by crushing in flat splinters instead of cubic form had to be regarded as damaging.

It was therefore considered as necessary carefully to wash with water the coarser stone balls (1½ in.), which the crushers had to pass, in order to free them from slime and clay sticking to them. In addition, the stone meal was removed from the broken material by means of passing this material over fine sieves. The milling of sand was entirely given up. Moreover, the crushed gravel was cleansed from the stone meal adhering to it by means of washing. These measures were in part carried out as a result of tests which had been made in the Swiss Testing Laboratory for Materials. These tests had as object the more careful examination and definition of the observations made at the place of construction.

The conclusions from these investigations are in short the following: The tests of the strength of the mortar and concrete gave the best results when the crushed sand was mixed in equal volume with pure round river sand. The most

favorable ratio of gravel and sand was found to be 2:1, both as regards compression and bending strength. For pure round sand a material supplied by the Entreprises Electriques Fribourgeoises was used.

This gravel and sand mixture was used for the large arches and the superstructures.

For the purpose of easier manipulation the addition of sand was raised somewhat above the most favorable strength ratio of 2:1. The gravel for this consisted of about equal parts of broken gravel and round river gravel. For the sand portion was taken: 2 volumes of round river sand and one volume of crushed sand, which was produced bit by bit in the breaking of the larger stones and then was mixed with an equal quantity of round river sand which was dredged at the bridge site. Pure broken sand therefore made up as a rule only one-sixth of the entire amount of sand which served to manufacture the concrete.

The actual addition of cement per cu. yd. of *finished* concrete amounted to 4, 6; 5, 6; 5, 7 bags, respectively. The general characteristic of the tests was that they showed more regular and satisfying results than earlier in the case of the concrete for the piershafts. We will determine the later hardening of this concrete by means of numerous concrete cubes.

Insofar as irregularities occurred in the tests, they are to be explained by the fact that despite all precautions to keep the mixture of the sand and gravel constant, this mixture, nevertheless, constantly changed. In addition, heat and cold as well as sun and rain exercised their unavoidable influence. Finally, the cement itself changes its characteristics slightly, so that in the case of a combination of unfavorable values, the extreme minima and maxima lie fairly far apart.

We did not fail to check up constantly on the granulometric composition of the gravel and sand; the results were regarded as sufficiently good.

After these investigations in reference to the *quality of the concrete*, everything seemed to be done to which we were obligated in regard to the great construction. The results of the tests can be regarded as satisfactory, inasmuch as highly plastic concrete with a slump of 4 in. and often more was used for the arches and superstructures. In the piers, as a rule, it amounted to 3 in., but also 4 in. occurred corresponding to the definition of a highly plastic or liquid concrete. When a contractor is once accustomed to use plastic concrete or liquid concrete, it is hard to withstand the temptation to thin the concrete in an increasing degree. The best rule is never to apply concrete wetter than is absolutely necessary for the work in hand. An increasing number of voices urgently warn against adding very much water to concrete and throw doubt upon the much praised greater subsequent increase of strength of liquid concrete. It is certainly true that too much water leaves hollows in the concrete, which weaken the concrete.

In addition to the above investigations, we instituted a number of others of slighter extent, of which brief mention will be made.

In order to protect the reinforcing of the large arches from rust during transportation and mounting, it was coated with cement milk which contained an addition of potassium bichromate equivalent to 3 to 5 per cent of the mixing water. This mixture of cement and water must be thin, and therefore cement and "bichromated" mixing water were combined in the ratio of 1:1. The protection

of the steel was extraordinarily good, and, as later checking-up of the bridge showed, the concrete adhered as well as it would have adhered to bare steel. This is not the case with pure cement milk because the coating becomes cracked, which is not the case with a potassium bichromate coating.

In addition, the phenomena of the release of heat during the setting of the cement were observed. For this purpose we made use of the usual thermometers and secured very interesting results. The temperature in the interior of the pier on the Berne side, which was accessible through interior mine chambers, rose about 88 deg. F. and only after three to four months sank to the temperature of the air, which thenceforth exercised its influence in a greatly moderated and at times delayed fashion.

An unusual circumstance occurred in the concreting of the large arch. In order to carry out the building program, the contractors wanted to finish the large arch by the end of the year 1925. Various circumstances had delayed the beginning of the concreting of the arches operations so that they could only be started at the end of October. Since the temperature in the fall of 1925 was very low and began to fall abruptly, arrangements were made as soon as possible to observe the temperature of the concrete. The experience obtained in this way was to serve as a basis for determining at how low a degree of temperature it was safe to do concreting. The addition of ingredients for the prevention of freezing was entirely avoided in order not to cause corrosion of the steel.

The daily temperatures until the middle of November were over freezing, then, however, they fell abruptly and in the middle of a cold period reached a minimum of 10 deg. F., which lasted till the end of the month. Also, on Dec. 5 and 18 there occurred periods of decided cold with temperatures as low as 0 deg. F. Arch number one was concreted under still relatively good conditions of temperature. Without at first thinking of concreting at very low temperature, the rule was made that work must be stopped when the heat of setting was not sufficient to keep the temperature of the concrete 50 deg. F. for 72 hours. In order to fulfill this condition as long as possible in very cold weather, it was determined with the concurrence of the contractors to heat the sand and gravel bins from below, to warm the mixing-water to 105 deg. F. to 120 deg. F., and finally to place open fires (coke ovens) under the concrete mixing machines. In this way it was possible at temperatures as low as 14 deg. F. to keep the concrete at about 50 deg. F. when it left the machine and to get it into the forms at 43 deg. F.; then with the development of the setting heat it was possible to fulfill the imposed condition, as the records show.

A single unforseen occurrence took place in the concreting of the first arch ring of the seventh arch. The concreting was always undertaken after the consideration of the weather prophecy. With a covered sky and a medium amount of frost the concreting was begun at eight in the morning at 28 deg. F. After a flurry of snow toward evening, the weather cleared up. A sharp wind began to blow and the temperature fell uninterruptedly to 0 deg. F. The concreting was continued through the night with great difficulty. As a result of a defect in one of the mixing machines a stoppage unfortunately occurred so that a new machine, which was cold, had to be put in use. As a result of this the concrete was of a

temperature of only 35 deg. F. after it had been placed; even the setting temperature was no longer sufficient to fulfill the above-mentioned conditions. In this arch damages from frost took place in the under portions, particularly along the colder Melan girders and on the surfaces of the main arch, where the frost could work with particular ease.

It is still to be mentioned that the first ring or layer of the main arch was most sensitive to the effect of cold since it possessed only the small thickness of 1.3 ft. at the crown, and 2.3 ft. at the springing. In the case of the second ring or layer, which makes up two-thirds of the thickness of the arch barrel, the circunstances were much more favorable. Likewise nothing was omitted to make success certain. The backs of the first arch-ring were well rubbed and cleaned with soda water. The second vault ring was only concreted when the concrete of the first ring was above freezing. In addition, the forms were left until the spring of 1927, and the arches were covered with cement sacks and in places also with straw. In arch 7 both over and under sides were covered with asphalted cardboard in order to retain the warmth. And so the concreting of the large arch could, despite the low temperature, be finished by the end of December, 1925.

On the whole, therefore, it may be said that concreting is practicable at temperatures down to 14 deg. F. if all preparations for the warming of materials and the retaining the warmth after placement are made. Interruptions of concreting in cold weather must be avoided; it is advisable, therefore, to keep a reserve concrete mixer warm. According to the above experiences, however, concreting should be stopped at temperatures lower than 14 deg. F., even when all the above mentioned preparations are made and when careful watch is kept.

From records the effects of the setting temperature are to be seen in the case of the superstructure which was erected this summer, although for the most part in relatively cool weather. The development of heat is, despite the rather inconsiderable mass of concrete, very significant.

I. CONCLUSION

In conclusion I give expression to certain ideas which occurred to me in the course of the reconstruction of the Grandfey Viaduct:

1. First, it is to be noted that preliminary investigations of gravel and sand for concrete may be misleading if the samples of material are not taken in the usual manner of their preparation for the job. In the Grandfey Viaduct, failure to do this was of no great significance because the piers are not subjected to very severe stresses. Even the reduced resistance to frost caused by the excessive stone meal is, although undesirable, nevertheless of no important influence. For the large arches and the superstructure, a satisfactory gravel-sand mixture was used, which consisted of three instead of two primary components (fine gravel, coarse gravel and sand). Citations have been given above of the danger of crushed gravel and milled sand. Particularly in the case of limestone, crushing produces much stone meal, which can reduce the effectiveness of the cement to an extraordinary degree. I am, therefore, of the opinion that in the first place the effect of particular kinds of stone upon the manufacture of concrete should be investigated, both in regard to the crushed stone and natural gravel. By this means it would

be possible to prophesy more exactly the essential characteristics of a concrete to be made out of a given aggregate. In this connection I will mention only two known phenomena. Broken Urner granite makes essentially worse concrete than round sand and gravel of the same material. Internal breaking down and the production of thin flat pieces by crushing are the explanation of this phenomenon. A related phenomenon was observed in the use of very hard Flysh stone in the construction of the Linth Bridge near Schwanden. Other unusual circumstances result from the use of jurassic limestone, the subsequent strengthening of which is extraordinarily great. The explanation of this may be the delayed emission of water by relatively porous kinds of such limestones.

Whoever looks through the literature on concrete is terrified by the immeasurable extent of the experimental results, from which, generally, nothing or everything that one wants to deduce can be deduced, of course, if one has time at all to look through it thoroughly, which probably is granted to few. Order and insight will only come when the chief kinds of stone are investigated singly and then in combination, in which operation the influence of the construction of the stone-crusher and sand-mill should also be tested. The investigations by Americans like Abrams, form the beginning of a clarification of the primary influences in the preparation of concrete. The thoroughness and completeness of such investigations by the Americans are the strength of their engineers.

2. A further circumstance, which afforded extraordinary difficulty at the place of construction, was the addition of water. Gravel and sand always contain after exposure to the weather and washing operations more or less water. Despite all attempts, no hard and fast rule could be given for the measurement of the addition of water. Finally, in order to avoid lengthy discussions, the slump test usual in America was adopted. The slump for the concrete was specified with an extreme limit of 2.5 in. and the addition of water was measured according to this. I do not wish to discuss more at length the questions having to do with the addition of water, and I only add that a future solution must lie either in using only dry or almost dry materials, or in inundating the sand, whereby the bulking of the sand would be eliminated. This question, too, would merit a thorough treatment by those interested.

3. Another circumstance worthy of consideration is that cold at the time of concreting is not to be feared as much as it usually is. The results of the cube tests corroborate this. These cubes were only stored for one or two days in boxes during the cold weather and thereafter were piled in the open. The strengths of the summer and winter tests of such cubes were the same. To be sure, appropriate measures had to be taken in order to assure the setting of the cement. In such cases we were in the fortunate position to be able to eliminate, or at least greatly to reduce the effects of the shrinkage; nevertheless, shrinkage is the most dangerous foe of concrete, which exercises the first and most destructible effects. Through it weak places are formed for the later appearance of ravages from frost and other influences. The rules of the manual of the Swiss Federal Rys. that concreting is to be stopped at 27 deg. F., are thus justified; this temperature could, however, be reduced in the case of large masses of concrete, where concreting may be carried on even at 14 deg. F. if proper preparations are made and *good* cement

is used. Concreting at lower degrees of temperature would be welcomed both in the interests of the workers and contractors, for it would make possible, in many cases, continuous work in summer *and* winter. Let it be once more emphasized that ingredients to prevent freezing were not used lest they damage the steel; at times of great cold, however, soda was added to the cleaning water after experiments had shown that soda acts as a rust preventative.

4. The reconstruction of the Grandfey Viaduct, as well as the viaduct duDay near Vallorbe represents a new procedure in execution, in its kind and method, an execution which deserves to be used in other suitable cases. The operation of reconstruction is characterized by the fact that each part of the construction was erected independent of traffic loads and their effect in producing deformations, and later the new construction parts were used for the bearing of traffic loads. The large arches, therefore, despite the necessity of keeping them light in weight, were constructed as massively as possible and the track was kept always in the middle of the bridge. A great advantage of the novel method of construction is that the substructure of an old bridge can be retained, whereby often very inconvenient tie-ups of lines can be avoided. The only condition is, that the old substructure must be, in general, in good condition. With this method of reconstruction the art of the engineer is essentially raised; like the doctor, he must retain what is existing as far as possible and turn it to further use.

In the reconstruction of the Grandfey Viaduct, no train could be omitted, even the night trains travelled regularly. It is necessary in such reconstruction to exercise a careful watch; moreover, an engineer must always be present who is able to judge with certainty the various circumstances of construction.

The question of the influence of jars upon the setting of the concrete is to be answered by the fact that no disadvantages could be ascertained. This result agress with the observations of Rabut (see *Le Genie Civil*, June 14, 1924). During the concreting of the arches and superstructures, the speed of the trains was reduced to 6 mi. per hour. During the concreting of the arcades it would have been advisable, as in the case of the main arches, to form three sections and to ram the connecting joints with mortar in order to make certain that the changes of form which result from every crossing of the train should do no damage.

5. The execution of the construction was not difficult, on the contrary every position was easily accessible from the iron bridge. Accordingly, no gantry scaffoldings were necessary. That the construction was much simpler than had been originally assumed, is proved by the short time of construction. The installation was begun on March 1, 1925; the large arches were concreted by the end of 1925, and the work of construction was finished by the end of 1926, that is, after 22 months, except for the supplementary works and improvements, which on account of the cold had to be postponed until the spring. Experienced construction firms had thought it necessary to demand three years for construction. The time of construction for the old viaduct had been over four years, that is, almost exactly double that of the present reconstruction. For future similar reconstruction, the present reconstruction offers good bases for judgment of the difficulties of construction, which consist chiefly in the fact that the train traffic and the work of construction interfere with each other.

6. In the above description of the modern method of construction it is proved that the use of reinforced concrete for the reconstruction of bridges can be extraordinarily economical. At first only one track was laid over the viaduct, and that in the axis of the bridge. This afforded the possibility of assuring the preservation of the reinforced concrete under relatively slight accidental loads, and to collect further experience with increasing load and afterwards with the construction of the second track.

The most notable result of the calculation of the Grandfey Viaduct must be considered the significant working together between the main arch barrels and the superstructures (arches), which in many existing structures has led to the formation of cracks. As I have elsewhere emphasized, the working together of parts of the structure should be given more attention. We are of the opinion that an attempt to avoid such working together should not be made by the insertion of *hinges*, which often is done only in the interests of simpler calculations. Hinges are *weak places*, and result in most cases in undesirable difficulties in construction and upkeep. For these reasons hinges were purposely avoided as much as possible in the Grandfey Viaduct.* In these instances where the pure calculation in accordance with the classical methods is impossible or so complicated that, practically speaking, no end is to be seen, precise experiments with the Beggs' deformeter apparatus employing a model give the necessary conclusions. This method is, without doubt, capable of great development.

In conclusion, be it noted that in the near future a thorough loading-test of the Grandfey Viaduct will be carried out. The appraisal of the results and the comparison with the results of calculations will form the subject of a special report.

The company which executed the whole work, according to our design and plans, was Messrs. Prador & Cie, engineers and contractors, of Zurich.

* In answer to a written inquiry sent by Geo. E. Beggs, Mr. Bühler has written further on this subject as follows: "You say that some of your engineers argue that the superstructure of a bridge should not be intentionally designed as continuous with the arch itself, because this may bring the reinforced concrete of the floor under tension. If properly designed, I believe that such structures are very good. The greatest difficulty is to determine the forces and moments (inclusive of the action of shrinkage, temperature, braking forces,. etc.) If they are well established and the dimensions sufficiently and thoroughly chosen, I cannot understand why a continuous structure cannot be as good as one which is intersected. The latter is a poorer design because it will be more flexible, and therefore more subjected to impact. The numerous joints, necessary to suppress the continuity, will considerably raise the price of the structure, and cause greater maintenance costs. Every joint is a weak point. Up to to-day, we have found no crack in the slabs and beams of the foot bridge."

Researches on Concrete Materials and on Plain and Reinforced Concrete.

Submitted by Committee E-3 on Research.

Introduction.

It was pointed out in our 1926 report that the primary function of this committee is to give the Institute each year as complete information as possible regarding the work of research laboratories and educational institutions in the field of concrete and concrete materials.

We believe that these annual reviews will be of greatest benefit to the members of the Institute if they are prepared in substantially the same form as the 1926 report. This will enable those desiring information on current or earlier researches to find it in the annual *Proceedings* of the Institute.

Our 1926 report gave the principal agencies engaged in researches on concrete and related subjects as well as the publications in which the results were published. Much of the information for this report, the second submitted by our committee, was furnished by the laboratories which reported to our committee last year, hence such lists are not repeated.

Much of the data for this report were secured as a result of two questionnaires sent to 209 testing laboratories in the United States and Canada classified as follows:

Commercial laboratories and individuals	15
Government and city testing laboratories	28
Railroad testing laboratories	31
State highway departments and commissions	46
Universities and colleges	89
Total	209

Brief summaries of the replies to our questionnaires constitute the greater part of our report and are given in the accompanying appendix.

Outline of Report.

The appendix to this report contains the principal researches carried out in the United States and Canada during 1926 on concrete materials and on plain and reinforced concrete. A list of suggested researches is also included. An important feature of the report is a list of outstanding papers and reports of researches on concrete which were published during

1926 in the United States and in foreign countries. The subject-matter of the appendix may be classified as follows:

I. RESEARCHES ON CEMENT.

II. RESEARCHES ON AGGREGATES.

III. RESEARCHES ON PLAIN CONCRETE.

Abrasion
Admixtures
Building units
Curing
Dams
Durability
Field tests and control
Flow under load
Freezing and thawing
Jigging and vibration
Pavements
Permeability
Poisson's ratio
Proportions
Retempering
Sea water
Strength:
  Compressive
  Flexural
  Tensile
Volume change
Waterproofing
Workability

IV. RESEARCHES ON REINFORCED CONCRETE.

Beams
Bond
Chimneys
Slabs

V. SUGGESTED RESEARCHES ON CONCRETE AND RELATED SUBJECTS.

Concrete materials:
  Cement
  Aggregates
  Plain concrete
  Reinforced concrete

VI. REFERENCES TO PAPERS AND REPORTS ON RESEARCHES PUBLISHED DURING 1926.

Compiled principally from:

American Concrete Institute *Proceedings.*
American Highways.
American Society Civil Engineers *Proceedings.*
American Society Testing Materials *Proceedings.*
Canadian Engineer.
Concrete.
Engineering Experiment Station Bulletins.
Engineering News-Record.
Engineering *Journal* (Canada).
Foreign Technical Publications:
  Comptes Rendus, Paris.
  Beton und Eisen, Berlin.
  Le Ciment, Paris.
  Stahl und Eisen, Düsseldorf.

Technical Papers, Dept. of Scientific and Industrial Research, London.
Tonindustrie Zeitung, Berlin.
Zeitschrift des Vereines Deutscher Ingenieur, Berlin.
Zement, Charlottenburg.
Highway Research Board *Proceedings*.
Highway Research *News*.
Public Roads (U. S. Bureau of Public Roads).
Technologic and Scientific Papers and Circulars of U. S. Bureau of Standards.
Rock Products.

SCOPE OF RESEARCHES UNDER WAY.

Cement.—Important researches on cement are under way at the U. S. Bureau of Standards which have in view the development of specifications and methods for testing cements. The comprehensive studies on the constitution of portland cement inaugurated at the Bureau in 1924 in co-operation with the Portland Cement Association have been continued.

Studies of volume changes in neat cement and mortar are being continued at the University of Michigan. Studies of the relation between strength of portland cement tested by standard A. S. T. M. methods and the corresponding concrete strength are being made by a number of state highway departments in co-operation with the U. S. Bureau of Public Roads.

An extensive investigation on fluid cement-water mixtures, which was begun in 1925 at the Structural Materials Research Laboratory, Lewis Institute, Chicago, was continued and supplementary tests were carried out by 12 laboratories in co-operation with sub-committee VII on Strength of A. S. T. M., committee C-1 on Cement. Further studies on this subject are now being made by 52 laboratories in co-operation with sub-committee VII.

Aggregate.—Studies of aggregate include such subjects as effect of grading on strength and yield of concrete, development of abrasion tests for gravel and methods for determining the durability and soundness of aggregates for concrete, effect of loam and shale in coarse aggregate, development of methods for determining quality of fine aggregates, and studies of the suitability of local aggregates for concrete.

Plain Concrete.—The majority of investigations under way are concerned with plain concrete. The question of admixtures, both chemical and inert, including coloring materials, is receiving attention. Studies of curing of concrete are under way by the U. S. Bureau of Public Roads and by several state highway departments.

One of the most important researches is the co-operative investigation of a 60-ft. arch dam near Fresno, California, which is being carried out under the auspices of Engineering Foundation; auxiliary tests on the permeability and other properties of concrete are being made at the University of California.

Studies of the durability of concrete exposed to alkali, sea water, and freezing and thawing, and studies of volume changes in concrete under various conditions of exposure are receiving much attention. There are a number of investigations under way relating to field tests and methods of controlling the quality of concrete on the job.

Strength tests of concrete building units were carried out by committee P-1 on Standard Building Units and committee P-6 on Concrete Products Plant Operation of the Institute and fire tests on concrete tile walls were made at the Underwriters Laboratories.

Researches relating to concrete pavements include such subjects as finishing, relation of subgrade characteristics to behavior of pavement, and the behavior of reinforcing steel in pavements.

Extensive tests are being made on the strength and other properties of concrete, covering such items as compressive, tensile and flexural strength, modulus of elasticity in tension and compression, Poisson's ratio, resistance of concrete to compression in one, two and three directions, etc. These investigations are yielding important new data. Other investigations of plain concrete include studies of flow under load, proportions, retempering, waterproofing, workability, etc.

Reinforced Concrete.—Relatively few investigations are being carried out on reinforced concrete. Among the more important may be mentioned those on web stresses in beams carried out by the U. S. Bureau of Standards and the University of Illinois and the test of a 309-ft. chimney at Duluth, Minnesota, which is under way by Committee S-1 of the Institute.

SUBJECTS FOR RESEARCH.

The appended list of subjects suggested for research was compiled from replies to the committee's questionnaire and affords some indication of the problems on which information is needed. A number of the subjects listed are now under investigation and data on these will no doubt be available during the coming year.

REFERENCES TO PAPERS AND REPORTS ON RESEARCHES.

References to papers and reports of researches on concrete and related subjects which were compiled from engineering journals published in the United States and in foreign countries are given under Part VI of the appendix. In compiling this list the aim has been to report only the more noteworthy articles.

The committee expresses its appreciation to the many research organizations and individuals who have co-operated by furnishing information.

H. F. GONNERMAN, *Chairman*,
F. E. RICHART, *Secretary*.

APPENDIX.

RESEARCHES ON CONCRETE MATERIALS AND ON PLAIN AND REINFORCED CONCRETE.

Many of these researches have not been completed; hence, in general, reports giving data of the tests are not available. Reference is made in certain instances to publication of reports. The conclusions stated are those of the investigator and do not necessarily represent the views of the committee.

I.—CEMENT.

Fineness.—(U. S. Bureau of Standards, Washington).

(a) Development of standard sieve series;
(b) Development of air analyzer for determining fineness.
(c) Preparation of standard samples for calibrating No. 200 sieves.

New Cements and Standard Methods of Testing.—(U. S. Bureau of Standards, Washington).

(a) Development of a constant temperature and humidity chamber for storing specimens;
(b) Specifications and methods of test;
(c) Procedure for comparative cement tests, particularly with reference to high early strength of portland cements.

Constitution of Portland Cement.—(Portland Cement Association Fellowship, U. S. Bureau of Standards, Washington). Continuing the exhaustive investigation of the fundamental chemistry of cement. Taking as a point of departure the investigations by Rankin and Wright on phase equilibria of the lime-alumina-silica system; studies are being made of phase equilibria of other systems made up of constituents in cement. The effect of various impurities normally present in raw materials for cement is being studied by their incorporation singly or in combination in a standard mixture of lime, alumina and silica. Studies of the mechanism of hydration and hydrolysis of individual cement compounds are being made. The optical constants of many materials have been determined and general improvements in petrographic methods of identification have been made. The application of X-ray methods as a means of identification is a recent addition to this work.

Volume Changes in Neat Cement and Mortar.—(University of Michigan, Ann Arbor). A study of the extent of changes and increase or decrease in magnitude with lapse of time. About 200 expansion bars from

many different synthetic and natural cements with known histories are available with measurements on all of them. Volume changes, with changes in moisture, do not decrease perceptibly for a period of at least 10 years.

Effect of Strength of Cement Upon Design of Concrete Mix.—(University of Kentucky, Lexington). Investigation to aid in design of a scientific mix. Seven and 28-day tests on concrete from 5 brands of cement having 7-day briquet strengths ranging from 210 to 400 lb. per sq. in.

Fluid Cement-Water as Criteria of Concrete Strength.—(Research Laboratory, Portland Cement Association, Chicago). A study of the possibilities of fluid cement-water mixtures as criteria of concrete-making properties of portland cement, in order to determine whether such mixtures: (1) more nearly measure the concrete-making value of cement; (2) give more concordant tests by different operators; (3) are more easily and cheaply made than standard mortar tests.

Other features of the investigation were: (a) comparison of strength of fluid cement-water mixtures with concrete strength using a wide range of hydraulic cements; (b) development of test for consistency of fluid cement-water mixtures; (c) selection of consistency best suited for strength tests; (d) selection of specimen best suited for strength test; (e) investigation of possibility of early strength test for portland cement at 1 or 3 days; (f) comparison of strengths of fluid cement-water mixtures at different ages with 1:3 standard sand briquets and cylinders.

A supplementary investigation of fluid cement-water mixtures was carried out by 12 laboratories in co-operation with sub-committee VII of A. S. T. M. committee C-1 on Cement, using samples of cement from the same lot and a special flow apparatus designed and built in this laboratory for determining the "basic water content." This supplementary investigation was designed to:

(1) Compare strength tests on fluid cement-water mixtures, standard sand cylinders and briquets made by 12 different laboratories using the same cement;
(2) Study the day-to-day uniformity of strength tests;
(3) Study the effect of varying the quantity of mixing water;
(4) Study the flow cylinder test for consistency;
(5) Study the possibilities of a 1- or 3-day strength test for portland cement.

The following additional phases of this study are now under way by sub-committee VII of A. S. T. M. committee C-1:

(1) Tests on a large number of different portland cements by 52 laboratories in order to establish proper strength requirements at 3, 7 and 28 days;
(2) Study of time of setting and soundness of portland cement using fluid cement-water mixtures.

Co-operative Cement Tests.—(Illinois Division of Highways, Springfield, in co-operation with American Association of State Highway Offi-

cials). Comparison of strength of portland cement by standard methods and strength of concrete. Six brands of cement were used, 2 low strength, 2 medium and 2 high strength. Briquets and 2 x 4-in. standard mortar cylinders were made according to standard methods. Parallel compression and transverse specimens were made using a 1 : 2 : 3½ mix and tested at 7, 14, 28 and 90 days, using standard and cantilever testing apparatus.

II.—AGGREGATES.

Comparison of Various Gradations of Stone and Gravel Aggregates.—(N. J. State Highway Commission, Trenton, in co-operation with National Sand and Gravel Association, National Crushed Stone Association, and U. S. Bureau of Public Roads). Study of the most economical grading and proportions. Thirteen gradings used covering the commercial range of stone and gravel. Proportions: Series A, N. J. State Highway 1 : 1¾ : 3½, flow constant; Series B, ratios sand to stone 33 : 67, 36 : 64 and 40 : 60, flow and water-cement ratios constant; Series C, proportioned by water-cement ratio and fineness-modulus method. Specimens 8 x 8 x 48-in. beams, and 6 x 12-in. cylinders. Age, 28 days and 6 mo. Results not available.

Effect of Grading of Aggregates on Strength of Concrete.—(Iowa State Highway Commission, Ames). Wide variations in grading of coarse aggregates due to differences in deposits and due to segregation by improper piling methods in handling materials, even from a single source, have caused much speculation as to the probable effect of variations in grading upon strength of concrete. An investigation of this factor has thus far indicated that variations in grading, except for improbable extremes, have a negligible effect upon strength of concrete made with proportions having even a slight excess of mortar in the mix. This has permitted a wider range of use in different proportions of materials from certain plants which had been limited largely to use in only one proportion.

Study of Quality of Two Types of Aggregates from Western Wisconsin.—(University of Wisconsin, Madison). Comparison of strength and durability of concrete with aggregates from Western Wisconsin in order to determine most efficient mix. Tests on one hundred and eighty 8 x 8 x 30-in. beams and two hundred and eighty-five 6 x 12-in. cylinders.

Abrasion Tests for Broken Stone and Gravel.—(U. S. Bureau of Public Roads, Washington). Investigation of various suggested modifications of Deval abrasion tests for rock and gravel in an effort to develop a test which will put the two materials on an equal basis so that identical specification requirements may be used.

Abrasion Test for Gravel.—(Illinois Division of Highways, Springfield). A study of modified forms of abrasion tests combined with sodium sulphate tests to determine amount of soft, unsatisfactory materials. Deval abrasion machine with slotted and solid barrels used. Tests to be continued.

Durability of Aggregates.—(Allegheny County Bureau of Tests, Pittsburgh). Study of suitability of aggregates for concrete. Some aggregates are definitely unsatisfactory for use in concrete, although a chemical analysis would indicate that they are not different from sound aggregates.

Soundness Tests of Aggregates.—(Illinois Division of Highways, Springfield). Determination of soundness of coarse aggregates by sodium sulfate method. Ten pieces immersed in saturated solution of sodium sulfate at 70° F. for 20 hr., then placed 4 hr. in drying oven at 100° C.; repeated 5 times and condition noted.

Soundness Tests of Aggregate.—(Research Laboratory, Portland Cement Association, Chicago). Soundness tests by alternate immersion in saturated sodium sulphate solution and drying at 105° C. (Brard's tests) on 30 aggregates representing a wide range of materials. Tests carried through 10 or more cycles.

Limestone Screenings as Fine Aggregate.—(University of Kentucky, Lexington). Compression tests of cylinders, etc., at 28 days and 1 yr. Screenings from first-class limestone were satisfactory, but screenings from Kentucky limestones are doubtful and have never been produced commercially so that they can be recommended.

Sandstone for Concrete Aggregate.—(University of Kentucky, Lexington). Study to find ways to use sandstone satisfactorily in concrete. Tests show that sandstone having less than 6 per cent wear can be successfully used in concrete pavement, and that this limit may be extended up to 10 or 12 per cent. Progress has been sufficient to cause highway department to construct 18 miles of concrete pavement using sandstone having 6 per cent wear for both coarse and fine aggregate.

Dirty Stone as Coarse Aggregate.—(University of Kentucky, Lexington). Effect of dirty stone on 28-day, 1, 2 and 3-yr. compressive strengths of concrete cylinders exposed to weather after 28 days. Stone showed 4.5 per cent loss by washing, about one-half of which was clay and silt, and one-half limestone dust. Judging from 28-day and 1-yr. tests, dirt appeared to have no harmful effect.

Study of Methods for Determining Quality of Fine Aggregates.—(U. S. Bureau of Public Roads, Washington). Investigation of proposed test for quality of concrete sands involves application of water-cement ratio principle which may be substituted for present unsatisfactory strength-ratio test.

24-hr. Strength Test for Sand.—(Kansas State Agricultural College, Manhattan, Kan.). Accelerated test for predicting strength of sand for concrete. Lumnite cement used for 24-hr. tests for comparison with portland cement strength-ratios at 7 and 28 days. Tension and compression tests compared with parallel tests on Ottawa sand. Seems probable that 24-hr. test will be satisfactory, but it remains to be investigated whether it will detect injurious organic matter.

Sampling and Handling Aggregates.—(University of Michigan, Ann Arbor). Investigation to improve sampling methods and determine effect of various methods of handling in stock piles. Results published in *Proceedings*, 1926 Conference on Highway Engineering, University of Michigan.

Louisiana Gravels as Aggregates.—(Louisiana State University, Baton Rouge). Studies to obtain data for proportioning for compressive strength.

Concrete-Making Materials of Arkansas.—(University of Arkansas, Fayetteville, in co-operation with state highway department). Study of extent, distribution and characteristics of natural sands, gravel and stone.

Effect of Shale on Strength of Concrete.—(Iowa State Highway Commission, Ames). Shale in very small amounts affected strength of concrete. Most serious effect arises from pitting of finished surface by spalling of concrete due to expansion of shale pebbles after concrete has set. To prevent an unsightly surface, as well as to avoid impairment of strength, quantity of shale must be kept small. Presence of rotten stone in aggregate affects strength less than the shale, but this material disintegrates when exposed to weather and crumbles under action of traffic. More than about 3 per cent by weight in coarse aggregate is likely to result in a speckled surface due to holes left by rotten stone after about 1 or 2 yr.

III.—PLAIN CONCRETE.

Abrasion.

Abrasive Resistance of Concrete.—(Kansas State Agricultural College, Manhattan, Kan.). A study of abrasive resistance of concrete as affected by various items, particularly the abrasive value of coarse aggregate. Study has been made of various brands of cement, cement hardeners, proportions and coarse aggregates having loss of from 2 to 20 per cent. Nine-in. concrete spheres tested in a standard brick rattler. Three spheres constitute a charge, 900 revolutions producing sufficient loss for comparative purposes.

Test furnished dependable means of measuring abrasive resistance of various concretes. Abrasive loss of concrete not seriously affected by abrasive value of coarse aggregate for aggregate losses of 8 per cent or less.

Admixtures.

Calcium Chloride as an Admixture.—(Iowa State Highway Commission, Ames). Studies by several investigators indicate that small amounts of calcium chloride when used as an admixture in concrete have effect of accelerating hardening so that relatively high strength is secured in a few days. If calcium chloride can be used without disadvantageous results, important applications are possible. If it is proper to use this method as a substitute for damp earth covering in curing of concrete road slabs, there are many obvious advantages to both contractor and engineer. Several miles of pavement, with calcium chloride as an admixture, are now being constructed in Iowa. The research organization has a special crew making tests of the quality of this concrete in comparison with concrete not so

treated and careful observation is made of all factors which appear to have a bearing on this problem. The laboratory is making parallel tests.

Effects of Lime in Concrete.—(National Lime Association, Washington). Preliminary investigation of methods of handling, mixing and preparing specimens approximately completed; bibliography prepared. Various concrete mixes handled and placed under conditions as nearly as possible simulating field conditions and requirements. No results available.

Effect of Celite in Concrete.—(University of California, Berkeley). Continuing study of effect of celite on elasticity and permeability of concrete.

Diatomaceous Earth as an Admixture in Concrete.—(Bureau of Standards, city of Portland, Oregon). Study of effect on strength of adding various percentages of diatomaceous earth to different concrete mixtures.

Tests of Portland Cement Colors.—(Research Laboratory, Portland Cement Association, Chicago). Continuing investigation of color value and permanence of 275 commercial colors in mortars upon exposure to the weather. Periodic inspections of specimens are made, color of each specimen being determined by comparison with Munsell discs. Parallel tests are made on specimens exposed to weather and on specimens from the same batches kept in the dark in laboratory. Tests of effect of mix, consistency, and curing condition on color, color permanence and compressive strength of mortars containing representative colors are included.

Building Units.

Tests of Concrete Masonry Units.—(Am. Concrete Inst. Comm. P-1 and P-6). Studies of effect of outdoor curing conditions on the strength and absorption of standard concrete building block were made in co-operation with the University of Minnesota and the Diamond Block Co., St. Paul, Minn. Data of tests will be included in the 1927 report of committee P-6 on Concrete Products Plant Operation.

A brief study was made of effect of various gradings of aggregates having same fineness modulus upon strength and absorption of concrete building tile manufactured by the dry-tamped process. Tests made in co-operation with Acme Concrete Products and Gravel Co., Cement City, Mich., and state highway laboratory, University of Michigan, Ann Arbor. Results will be included in 1927 report of committee P-1 on Standard Building Units.

Fire Tests on Concrete Building Tile.—(National Stone Tile Corp., San Francisco). Standard fire tests were made at the Underwriters Laboratories, Chicago, on "stone tile," a concrete masonry unit manufactured by a patented process of the National Stone Tile Corp, San Francisco. Siliceous aggregates were used in making some of the tile. Walls 8 and 12 in. thick were tested under standard fire test conditions and received 3 and 4½-hour ratings, respectively.

Curing.

Curing Concrete Pavements.—(U. S. Bureau of Public Roads, Washington). Investigation involves study of large number of concrete slabs, 2 ft. wide and 200 ft. long cured with wet earth, wet straw, calcium chloride (surface application) calcium chloride admixture and sodium silicate (surface application). Effect of reinforcing on distribution of transverse cracks also being studied.

Curing Concrete.—(Minnesota Highway Department, St. Paul). Study of relative efficiency of calcium chloride admixture and surface application, and air and earth curing. Included compression tests on concrete cured by integral calcium chloride, air and wet earth, and flexural tests on concrete beams cured by integral calcium chloride, surface application of calcium chloride, air and moist earth and tested as a cantilever. Ball tests made on mortars cured in 4 ways mentioned.

Calcium Chloride as Curing Agent.—(University of Michigan, Ann Arbor). Study of effect of varying quantities of calcium chloride on surface of concrete. Calcium chloride placed on blocks of concrete which were later studied for depth of penetration of the calcium chloride, deterioration of surface, and wear in Talbot-Jones rattler. Results published in *Proceedings*, 1926 Conference on Highway Engineering, University of Michigan.

Dams.

Arch Dam Investigation.—(Committee on Engineering Foundation, New York). Co-operative investigation of a single-arch type concrete dam near Fresno, Calif. Dam has constant radius of 100 ft., a thickness of 7½ ft. at the bottom, of 2 ft. for the upper 30 ft., and a height of 60 ft.

Measurements of strains, deflections and temperatures made at numerous points on dam with specially designed apparatus. Thus far, routine of tests has consisted of: (1) Measurements of dam under conditions of no-load; (2) loading the dam by admitting water to pond behind it up to a definite predetermined elevation and holding the water at that elevation while a complete series of measurements were being made; (3) withdrawing water from behind dam until a subsequent test; (4) repeating these tests so as to secure complete check measurements. Duplicate tests have been made with water at 20, 30, 40, 50 and 60-ft. elevations.

Heavy rain storms in November gave the dam a severe test, causing a head of water 3 ft. higher than the crest of the dam, filling the reservoir with rock, sand and silt to a depth of 40 ft. (or two-thirds the height of the dam), and probably causing a considerable increase of pressure above that of the water. On January 5th the water was still running over the top of the dam to a depth of 1 inch, and the increase in deflection, since the previous reading in September under the same head was 0.1 inch at the crown. The dam showed no signs of distress.

During the routine of tests the load readings followed within a few hours after the no load readings, and all tests were made at night. By

these means temperature effects on the results were eliminated, and there was not sufficient time under load to permit an appreciable amount of "flow" of the concrete. Since a flow of water over the dam at all times was maintained, the effect of change in moisture was eliminated, at least to considerable extent. By eliminating the above disturbing elements, the observations give the effect of the load in causing deflection and strains. It has been possible to determine approximately how much load has been carried by arch action, by horizontal bending, and by vertical bending respectively.

See Bulletins 1 to 4, Engineering Foundation Arch Dam Investigation, Eng. Soc. Bldg., New York City.

Tests of Concrete Composed of Same Materials Used in Construction of Stevenson Creek Test Dam.—(University of California, Berkeley). Tests being made for committee on arch dam Investigation of Engineering Foundation. The purpose of the tests is to determine physical properties of concrete of which the Stevenson Creek test dam is composed. Tests consist of determinations of modulus of elasticity and Poisson's ratio for various ages and for loads repeated at intervals; changes in linear dimensions due to variation in moisture conditions and causes other than variations in temperature and stress; flow or plastic deformation occurring under various stresses for varying periods; coefficient of thermal expansion and effect, if any, of age of concrete and moisture content; permeability, varying head, age and time.

Investigation of Materials to be Used in Mt. Morris and Caneadea Dams.—(Henry L. Howe, Testing and Consulting Engineer, Rochester, N. Y.). Study to determine best and most economical methods of using materials available for construction of these dams.

Fourth series of tests to be started this winter to determine relative value of available brands of portland cement in combination with materials available for construction of Caneadea dam. Permeability tests will be made on concrete mixtures with and without high natural silt fillers in the test specimens.

Durability (See also Freezing and Thawing).

Factors which Influence Disintegration of Concrete.—(Board of Water Supply, New York City). An investigation having for its purpose elimination of uncertainty of result in concrete quality, with view to securing maximum permanence and durability under all conditions of exposure.

Large and small specimens studied under various conditions of exposure. Includes study of texture and structure of good and bad concrete; evaluations of factors which influence texture and structure, including differences between cements and between different samples of same cement; constitution of cement with particular reference to quality of hydration products as influenced by completeness of calcination as well as effect of its alkali content; effects produced on cement by soluble constituents of

sand and coarse aggregates; effect of different clays; effect of temperature on texture and structure.

Durability of Concrete in Alkali Water.—(Kansas State Agricultural College, Manhattan, Kan.). Study to determine cause and method of prevention of disintegration of concrete in alkali water. Laboratory specimens stored in alkaline solutions under observation nearly 2 yr.; field specimens placed in suspected streams over 1 yr. Includes survey of structures in suspected territory. Has developed into study of soundness of aggregates.

Disintegration has been almost wholly caused by use of (a) an unsound coarse aggregate (limestone); (b) very lean and pervious mixes.

Development of Concrete that Will Resist Action of Alkalis.—(U. S. Bureau of Public Roads, Washington). Study of behavior of large number of specimens made and cured in various ways from different cements when subjected to action of alkali in laboratory and in the field.

Methods of Protecting Concrete Against Action of Alkali and Salt Water.—(U. S. Bureau of Public Roads, Washington). Specimens of concrete which have been treated with number of priming coats of water gas tar followed by a seal coat of heavier tar product are being subjected to action by alkali, both in laboratory and in the field, and their relative behavior noted.

Effect of Acid Soils on Concrete.—(U. S. Bureau of Public Roads, Washington). Specimens of concrete of known quality are being placed in acid soils and results noted.

Durability of Concrete.—(Hydro-Electric Power Commission, Toronto, Canada). Continuing investigation covering a study of factors governing resistance of concrete to action of frost and other weathering agents with the object of learning how to make concretes resistant to these agents in an economical manner.

Field Tests and Control.

Control of Water Content of Concrete Mixtures.—(Iowa State Highway Commission, Ames). It is a generally accepted fact that for uniformity, a definite ratio of water to cement should be maintained. On concrete pavement construction, due to variable moisture content of aggregates, speed of operation, large quantities of materials handled per batch, variation in water measuring devices, and a measurement of materials miles away from the mixer, keeping a constant water content presents many difficulties. Studies made during the past two seasons indicate that consistently accurate water measuring devices are a possibility, and that much improvement can be made in the control of the other factors.

Investigation of Accuracy of Volumetric Field Method of Proportioning by Void Bulkmeter Apparatus.—(Illinois Division of Highways, Springfield). Study to check volume, gradation, voids, unit weight and

compaction of aggregates during handling and the bulking of fine aggregates. Batch of coarse aggregate placed in void bulkmeter, initial reading of volume taken and batch then hauled 6 miles and amount of settlement determined at end of each mile. Voids determined by water displacement method. Entire batch then sieved. Batch of fine aggregate hauled same as coarse aggregate then inundated at end of 6 miles for relation of settlement due to haul and that obtained by inundation.

Average variation of actual field volumes of coarse aggregate at 40 proportioning plants was within 2 per cent of theoretical volume; average variation of actual field volumes of fine aggregate showed a deficiency in the sand of 8 per cent based on the inundated volume and not on the resulting volume due to haul; use of compacted volumes of materials to conform to 1 : 2 : 3½ volumetric proportions gives an increase in transverse strength over strength obtained under present field methods; workability of concrete is more affected by a deficiency of sand in angular than in rounded material.

Making Concrete Entirely by Water-Cement Ratio.—(Barney-Ahlers Construction Corp., New York City). Study to obtain concrete of maximum uniformity of strength and as near required strength as possible. Proportioning water exactly to amount of cement, mechanically; deducting mechanically from water tank estimated amount of water contained in aggregate; controlling consistency of concrete by amount of aggregate used; establishing "Job Constant of Strength" which fixes water-cement ratio for any type (strength) of concrete.

The Ahlers Concrete Strength Regulator, which balances each batch of water against cement automatically and insures constant water-cement ratio between batch and batch, making possible exact water-ratio control without recourse to calculations, was used.

Water-ratio control possible for field use; results from concrete made under this method of measuring the water and cement are remarkably uniform; concrete can be placed more easily and economically.

Prehydration of Cement in Concrete.—(Barney-Ahlers Construction Corp., New York City). Study to determine whether for the same water-cement ratio and mixing time, a stronger and more plastic concrete can be produced.

Water proportioned exactly to amount of cement, the two then passed into the prehydration unit and thence by gravity to the mixer drum, the required consistency of the concrete entirely controlling the amount of aggregate used per batch. The Ahlers Concrete Strength Regulator used in conjunction with an ordinary dry-powder-mixer, the latter as the means of mixing the cement and water.

Field Tests of Cylinders and Beams.—(Vermont State Highway Board, Montpelier). Field tests of 6 x 12-in. cylinders and transverse tests of 6 x 6-in. concrete beams made to check quality of concrete in pavements.

Flow Under Load.

Flow of Concrete Under Continuous Compressive Stress.—(University of California, Berkeley). Investigation of plastic deformations which occur in concrete when subjected to constant compressive stress over a period of time. Four x 14-in. concrete cylinders with strain gage at the third points of circumference. Strains measured periodically over 10-in. gage length, the compressive stress being kept constant by means of a car spring. Strains being measured with modifications of Whittemore gage, same as that used at Stevenson Creek test dam.

Other things remaining equal, plastic deformation which occurs with time is greater for lean mixes than for rich ones and greater for dry specimens than for wet ones. Preliminary experiments made last year indicate that specimens which are kept air dry continue to flow over a much longer period than for those kept wet.

Effect of Age on Plastic Flow of Concrete.—(Engineering Experiment Station, Ohio State University, Columbus). Continuing investigation to obtain reliable data for estimating stresses in concrete structures, such as arches, resulting from removal of forms early or late, and secondary stresses in building columns and beams. Also to furnish more test data on plastic flow in general. Four x 4 x 25-in. plain concrete specimens were placed in the compression side of a beam-like apparatus at different ages and strain gage readings were taken on all four sides daily as well as on an idle specimen in the same room and on an immersed specimen.

Freezing and Thawing (See Durability).

Study of the Resistance of Concrete to Repeated Frost Action.—(U. S. Bureau of Public Roads, Washington). Investigation under way for over a year, and it will be another year before any conclusions of value can be drawn. Involves the freezing and thawing of concrete beams from various types and grades of crushed stone, gravel, and blast-furnace slag coarse aggregate. General condition of concrete specimens and their strengths determined at various stages.

Freezing and Thawing Tests on Concrete Beams.—(Illinois Division of Highways, Springfield). Study of relation of effects of freezing and thawing on coarse aggregate and on 1 : 2 : 3½ concrete made therefrom. Sample frozen for 44 hr., then thawed in water at 80° F. for 4 hr. Cycle repeated 25 times. Freezing box as specifield by A. S. T. M. for tests of drain tile used. Found that there was no appreciable disintegration within number of cycles used.

Freezing and Thawing Tests on Rock and Crushed Stone.—(Illinois Division of Highways, Springfield). Study to determine effect of freezing and thawing on rock and crushed stone. Sample frozen 20 hr. at approximately zero temperature, then immersed in water at 70° F. for 2 hr. and heat applied to maintain original temperature of water, then weighed with surface dried and refrozen for 20 hr.

Length of Time Necessary to Protect Concrete in Cold Weather.—(Minnesota Highway Department, Duluth). Study of effect of frost, repeated freezings, etc., on concrete. Six x 6 x 30-in. beams cast of 1:2:3½ concrete and exposed to freezing temperature at different ages and for different periods; remainder of curing in dry air. Beams tested in cantilever machine.

Concrete frozen before set, thawed out and cured will not attain usual strength; freezing before set does most harm; 3 days' protection at room temperature appears to be sufficient.

Jigging and Vibration.

Effect of Jigging During Setting Period on Strength of Concrete.—(University of California, Berkeley). Continuing investigation in which over 800 cylinders have been tested to date. During the tests, rate of jigging has been 300 r.p.m. and time of jigging varied from 10 minutes to 5 hr. To date, conclusions are that effect of jigging greatly depends upon gradation of aggregate and that a large increase in strength is obtained when gradation of aggregate approaches that given by Fuller's curve with a slight excess of fine material. Where aggregate is not well graded, effect of jigging is to produce separation, excess size coming to top, with an accompanying decrease in strength. Tests being made using an "ideal mixture" and varying rate of jigging and height of drop. Indications are that with this "ideal mixture" an increase in compressive strength of more than 100 per cent over that of unjigged concrete can be obtained.

Pavements.

Investigation of Vibrolithic Process for Finishing Concrete Pavement.—(Iowa State Highway Commission, Ames). Comparison of Vibrolithic and hand finishing is being made. Thirty-three pavement slabs 7 ft. x 11 ft. x 8 in. thick were made and are being sawed into 363 beams 8 in. x 12 in. x 7 ft. for transverse tests. This is an extension of preliminary work done in 1923 when 625 beams of same size were tested. Earlier beams were molded in this size and not sawed from slabs, as in present series.

Pavement Core and Subgrade Survey.—(Colorado State Highway Department, Denver). Investigation to determine condition of subgrade under pavement, strength and characteristics of concrete pavement, and relation to condition of pavement. Calyx core drill cuts 4½-in. core from pavement. Subgrade sample tested for grading, clay content, moisture equivalent, lineal shrinkage, vertical capillarity and moisture content. Core is measured, class of concrete noted, tested for compressive strength, and specific gravity determined. Approximately one-half of paving in state has been covered by survey. Results not yet compiled.

Correlation of Soil Characteristics and Soil Tests with Behavior of Pavement Slabs.—(Iowa State Highway Commission, Ames). Involves countless soil tests, thorough surveys of pavement condition and careful

study of construction records. Although this work has been under way for several years, magnitude of task is such that it is barely started.

Crack and Joint Maintenance, Concrete Pavements.—(Iowa State Highway Commission, Ames). Laboratory investigations on comparisons of crack-filling materials, and correlation with service conditions.

Reinforcing Steel in Pavement Slabs.—(Iowa State Highway Commission, Ames). Continuing study of behavior of reinforcing steel in pavement slabs which has for sometime been subject of much conjecture. Opinions have been expressed that fracture of longitudinal rods would occur at transverse cracks. From work accomplished during past year, it is quite evident that fracture of rod is practically impossible in the case of smooth structural grade steel. Deformed bars and smooth bars are being studied.

Permeability.

Permeability of Concrete and Cement Mortars.—(University of California, Berkeley). Type of apparatus similar to that employed in some European experiments designed primarily for determining permeability of concrete from Stevenson Creek Test Dam. Concrete specimens are 18-in. diameter discs held between two flanges with rubber gaskets on either side between specimen and flanges. Water pressure applied to upper surface of specimen and water which passes through specimen is caught by 6-in. funnel clamped to lower face. Cement mortar specimens are 12 in. in diameter and a 4-in. funnel is used with these specimens.

Studies are also being made to determine relative efficiency of various integral waterproofing compounds.

Poisson's Ratio.

Modulus of Elasticity and Poisson's Ratio.—(University of California, Berkeley). Continuing investigation of effect of gradation of aggregates, cement ratio and age upon modulus of elasticity and Poisson's ratio.

Lateral deformations measured with modifications of apparatus used by A. N. Johnson; axial deformations measured with mirror extensometer.

Modulus of elasticity and Poisson's ratio increase with age. Poisson's ratio greater for low unit stresses than high ones. Poisson's ratio within ages so far studied ranges from 0.10 to 0.24.

Proportions.

Economics of Concrete Mixtures.—(Hydro-Electric Power Comm., Toronto, Canada). A continuing investigation to develop data whereby probable best proportions for any given set of aggregates can be approximated closely in advance and also whereby effect on cost of screening, regrading or other changes in the aggregates can be determined so that their advisability may be judged. Standard methods used in compression tests of concrete. Tests cover wide range of mixtures having different cement contents, consistencies and gradation of aggregates.

Method of Analysis to Determine Original Proportions of Mix.—(Iowa State Highway Comm., Ames). During 1925 an exhaustive study of several methods proposed by various investigators was completed. None of the methods used alone has been satisfactory for a general case, although each method is quite accurate for the particular set of conditions used by each investigator. In studies of uniformity of concrete of known proportions and in work with old concrete of an unknown mix, it is desirable to be able to determine the exact proportions of mix in the concrete of a given sample. Study led to suggestion of method making use of microscopic projections of polished surfaces, but the method has not yet been given a thorough trial.

Proportioning of Materials.—(Iowa State Highway Commission, Ames). On all paving projects proportions of materials for concrete are specified on a weight basis, and in proportioning they are actually weighed, insuring a constant relationship between quantities of the various ingredients of the concrete mix. In order to effect the most economic use of materials available, a series of proportions designed to produce concrete of equivalent strength has been computed for use with the weight method of proportioning. Specimens made in the laboratory or taken from the finished pavement furnished ample proof of correctness of theory of proportioning used.

Retempering.

Tests of Retempered Concrete.—(Research Laboratory of the Portland Cement Association, Chicago). The purpose of this investigation is to determine the effect of retempering and prehydration on workability of concrete up to 6 hr. after mixing and on strength at ages of 1 day to 1 yr. The investigation includes a study of: (1) Strengths and workabilities of 1 mortar and 3 concrete mixes after standing 0 to 6 hr. after mixing for a fairly wide range of water-cement ratios; (2) amount of water required to restore concrete to its original condition of workability (as measured by the flow test) after standing 2 to 6 hr. after mixing and the resulting concrete strength; (3) effect of type of aggregate on the strength and workability of concrete retempered up to 6 hr.; (4) effect on workability and strength of adding the aggregates to cement pastes after standing for various periods after mixing, for 3 mixes; (5) effect of absorption of aggregates on strength and workability of retempered concrete for 3 mixes; (6) effect of admixtures of hydrated lime and celite on the strength and workability of concrete retempered with and without the addition of water up to 6 hr.; (7) effect of premixing cement and water for periods up to 30 minutes before mixing with aggregate for one-half to 10 minutes.

Sea Water.

Tests of Concrete in Sea Water.—(Navy Department, Bureau of Yards and Docks, Washington). Continuing studies to determine best concrete mix, surface treatment, materials, cement and admixtures, for resistance to

disintegrating effect of sea water. Tests being carried out at Portsmouth, N. H., Charleston, S. C., at Pearl Harbor, T. H., and in Puget Sound, Washington. See "Tests on Concrete Piles in Sea Water Undertaken by Navy," *Eng. News-Rec.*, Sept. 2, 1926.

Effect of Sea Water Tide Action on Reinforced Pipe.—(South Carolina State Highway Dept., Columbia). Study to determine best method of protecting reinforcing wire against corrosion. Eighteen joints of 18-in. diameter machine-made pipe and 18 joints of hand-poured pipe were placed between high and low tide in sea water at Charleston, S. C. Two joints of each type are removed each year for examination and test.

Concrete pipe specifications should carry clause limiting absorption of concrete to 8 per cent (preferably 7 per cent); hydrostatic clause should be added limiting number of leaks through pipe walls, especially for the thin-walled 15 and 18-in. pipe.

Strength-Compressive.

General Concrete Investigation.—(U. S. Bureau of Standards, Washington, D. C.). Effect on concrete strength at different ages of (1) variation in cements; (2) gradation of coarse aggregates; (3) ratio of fine to coarse aggregates when coarse is deficient in certain sizes; (4) different types of aggregates.

Relation Between Strength of Portland Cement as Determined by Usual Routine Test and Strength of Resulting Concrete.—(U. S. Bureau of Public Roads, Washington, D. C.). Tests under way will be continued in the laboratory of the Bureau and in co-operation with various state highway departments. Indications are that there is fairly consistent relation between tensile strength of portland cement briquets and strength of concrete in which cement is used. Relationship holds true up to and including 28-day test.

Tests of Lumnite Cement Concrete.—(Washington University, St. Louis). Determination of water-cement ratio-strength curve, and effect of storage upon properties of lumnite.

Water-Ratio-Strength—Relation for Mortars.—(Research Laboratory, Portland Cement Association, Chicago). Investigation of effect of: (1) grading of aggregate on water-ratio-strength relation for sand mortars; (2) absorption of aggregate on effective water-cement ratio; (3) workability of mortar on water-ratio-strength relation.

Elgin sand of five gradings ranging from fine to very coarse, standard Ottawa sand and Elgin sand screened to same size as standard Ottawa sand, used in 5 mortars ranging from 1:1 to 1:3. Each mortar carried through 4 or 5 water-cement ratios varying by ½ gal. per sack of cement. Tests made using both dry and inundated sand. About 975 6 x 12-in. cylinders and 1950 2-in. cubes tested in compression at 28 days.

Resistance of Concrete to Compression in One, Two and Three Directions.—(T. & A. M. Dept., Univ. of Illinois, Urbana). Study of laws of

failure of concrete in compression with applications to manner of failure of spirally reinforced-concrete columns.

Tests of concrete cylinders subjected to liquid pressure in two directions, and loaded in third direction in a testing machine. Concrete encased in thin sheet brass or rubber envelope. Oil pressure cylinder of 7,000 lb. per sq. in. capacity was designed for tests. Special extensometers were built for strain measurements and special diameter gage for determining Poisson's ratio for columns.

Study of Effect of Methods of Placing Concrete.—(University of Illinois, Urbana). Study of strength, density and workability of concretes of various water contents and amounts of coarse aggregate when given different degrees of compaction during placing.

Compression tests of 6 x 12-in. cylinders and 3½ x 7-in. cores cut from slabs. Concrete placed by three methods. Much of the theoretically available high strength of dry mixtures may be lost by careless or unsystematic methods of placing. This is particularly true of mixtures with high proportion of coarse aggregate.

Strength-Flexural.

Comparison of Transverse and Compressive Strength.—(University of Wisconsin, Madison). Study to determine effectiveness of 6 x 8 x 30-in. and 4 x 6 x 24-in. beams as compared with 6 x 12-in. cylinders as measures of quality of concrete. Two hundred and seventy cylinders and 270 of each size of beams were made and considerable portion have been tested. Transverse tests made with lever machine on specimens supported as cantilevers; two breaks obtained on each specimen.

Comparison of Transverse and Compressive Strength of Concrete.—(Illinois Div. of Highways, Springfield). Present design formula for pavements uses transverse rather than compressive strength of slab as limiting factor; non-uniformity of compression tests on cores casts doubt on their value. Compression tests made on specimens drilled from beams on which transverse strength tests were made. Cantilever testing apparatus used.

Variations in results of transverse strength tests are only one-third of that found in compression tests. Ratio of transverse to compressive strength is not constant but varies with age.

Transverse Testing of Road Materials.—(Allegheny County Bureau of Tests, Pittsburgh). Study to determine earliest date at which highways may be opened. Beam testing machines used on each road job and data obtained to determine comparative values of modulus of rupture and compressive strength for various methods of curing. Specially designed beam machines used for testing 6 x 8 x 36-in. beams as cantilevers, making two breaks per specimen.

Seems reasonable to believe that in summer it is unnecessary to keep road closed more than few days, except to extremely heavy traffic or trucks.

Study of Method of Loading in Flexural Tests of Concrete.—(Research Laboratory, Portland Cement Assn., Chicago). Tranverse tests made on 520 7 x 10 x 38-in. concrete beams and parallel compression tests on 110 6 x 12-in. concrete cylinders at 3, 7 and 28 days, 3 mo. and 1 yr. Tests made in co-operation with A. S. T. M. committee C-9 on Concrete and Concrete Aggregates to determine size and shape of beam and method of loading which gives most uniform and satisfactory results.

Strength-Tensile.

Concrete in Tension.—(University of Maryland, College Park). Comparison of strength in tension to strength in compression. Special apparatus used consisting of molds for casting tension specimens and brass end clips for holding specimens in testing machine. For mortars and concretes less than 90 days old, ratio of tensile to compressive strength is practically the same for mortar and concrete, but varies with age from 15 to 18 per cent. For all mortars and concretes over 90 days old, ratio was about 8 per cent and constant whatever the age.

Study of Concrete in Direct Tension.—(U. S. Bureau of Public Roads, Washington, D. C.). Investigation to determine factors which influence tensile strength of concrete, and to establish relation which tensile strength bears to other physical characteristics of concrete. Effect of type of coarse aggregate on tensile strength specially investigated.

Relation Between Strength of Cement and Strength of Concrete.—(State Highway Laboratory, University of Michigan, Ann Arbor). Study to determine effect of variations in tensile strength of cement upon tensile and compressive strength and modulus of rupture of concrete. Tests made on various specimens up to and including 28 days. Tension tests on 6 x 12-in. concrete cylinders. "Thumbometer" used in standardizing briquet making. Special clamps used for tension tests of concrete cylinders.

Tension Tests of Concrete.—(Research Laboratory, Portland Cement Association, Chicago). Study to determine effect of following factors on tensile strength: (1) size, grading and type of aggregate; (2) quantity of cement; (3) quantity of mixing water; (4) age of concrete; (5) curing of concrete; (6) area in tension; (7) length of specimen. Parallel compression tests made on 6 x 12-in. concrete cylinders and transverse tests on 7 x 10 x 38-in. concrete beams for variety of conditions to obtain data for comparing tensile, compressive and flexural strength of concrete.

Investigation comprises tension tests on about 980 2 x 6-in., 6 x 12-in., 6 x 18-in., 6 x 24-in., 6 x 30-in. or 10 x 30-in. cylinders; compression tests on 770 6 x 12-in. concrete cylinders and transverse tests on 845 7 x 10 x 38-in. concrete beams at 3, 7 and 28 days, 3 mo., 1, 2 and 5 yr.

Deformation measurements were made with a mirror extensometer of Martens type to obtain modulus of elasticity of concrete in tension and compression.

Volume Change.

Expansion and Contraction of Concrete Due to Changes in Moisture Conditions.—(University of California, Berkeley). Study of the elongation and contraction accompanying variations in moisture conditions, including variations in atmospheric humidity. To date, experiments have been made to determine contraction of concretes under conditions wet to oven dry and the expansion from an oven dry condition to a state of moisture equilibrium at various humidities. Tests are now being inaugurated to determine contraction from a condition of complete saturation to a state of moisture equilibrium at various humidities. Specimens are stored in a specially constructed room where it is possible to control the temperature and humidity. Devices are installed for replenishing the air with moisture as well as removing moisture from the air. The specimens are 3 x 3 x 40-in. with steel contact points in the ends. Changes in length are observed with a specially constructed extensometer.

Other things remaining equal, the change in length varies with the cement ratio and also with gradation of aggregate, being influenced particularly by the character of the very fine particles of aggregate. The indications are that materials in which the very fine material is clay have a much greater influence on the expansion and contraction of concrete than do sands for which the very fine material is of crystalline structure.

Thermal Expansion of Concrete.—(University of California, Berkeley). Tests to be made on specimens 3 x 3 x 40-in. with a range in temperature from freezing to about 115° F. The specimens will be stored in a room in which the temperature can be varied at will between these limits. The purpose of the tests is to determine the coefficient of thermal expansion of concretes of various ages and for concrete in both a dry and saturated condition. For these tests a special extensometer has been devised which is compensating for changes in temperature.

Investigation of Expansion and Contraction of Mortars and Concretes.—(U. S. Bureau of Standards, Washington). Work has been extended and study is being made of the effects of amounts of water, quantity of cement and gradation of aggregates on expansion and contraction of mortars and concretes when applied to different types of bases.

Volume Changes in Concrete.—(Research Laboratory, Portland Cement Association, Chicago). Purpose of this investigation is to determine nature and cause of shrinkage and expansion of concrete and to develop a means of reducing or eliminating these changes. In developing a method for measuring contraction and expansion, four types of instruments were studied: (1) line comparator; (2) end comparator; (3) Berry strain gage; (4) Slater strain gage. Specimens representing a considerable range in cementing materials, manufacturing conditions and curing are under observation in order to obtain preliminary data upon which to base a comprehensive investigation of this important subject.

Waterproofing.

Tests on Efficiency of Penetrative Surface Treatments for Waterproofing Concrete.—(University of Wisconsin, Madison). Comparison of leakage through concretes of various densities before and after treatment with various penetrative surface treatments for waterproofing and determination of weathering properties of such treatments. Preliminary tests on 15 common types of surface treatments. Tests show that several preliminary treatments have considerable efficiency when first applied, but there is considerable variation in efficiencies of different treatments. Bulletin 1245 of Eng. Series, University of Wisconsin, on "Permeability Tests on Broken Concrete," by Withey and Wiepking describes methods employed.

Workability.

Study of Workability of Concrete Mixtures.—(U. S. Bureau of Standards, Washington). Development of apparatus which will measure workability of concrete and determine effects of admixtures on properties of concrete.

Paste-Ratio as Criterion of Workability of Concrete.—(Research Laboratory, Portland Cement Association, Chicago). Study made of effect of amount of cement-water paste on workability of mortar and concrete mixtures. Earlier tests showed that workability was, to some extent, measured by ratio between absolute volume of cement paste and space surrounding the aggregate. Since absorption of water by aggregate during mixing has an important influence on volume of paste produced, supplementary tests were made to determine rate of absorption during mixing. Investigation included compression tests of 1,030 6 x 12-in. concrete and mortar cylinders at 28 days.

Tests developed fact that due to difficulties of measuring waste materials it was not possible to determine exact amount of paste entering into a specimen with sufficient accuracy to make it a useful criterion. Tests indicated that for all mixes, except those in which aggregate was largely fine material, true workability is not produced until aggregates are thoroughly incorporated in cement paste. That is to say, for aggregates with usual proportions of coarse material, a truly plastic mix is without honeycomb.

IV.—REINFORCED CONCRETE.

Beams.

Investigation of Reinforced Concrete.—(U. S. Bureau of Standards, Washington).

(a) Shear tests of reinforced-concrete beams. (See Tech. Paper 314, by Slater, Lord and Zipprodt.)

(b) Tests of reinforcing bars rolled from new billets and from scrap steel.

Investigation of Web Stresses in Reinforced-Concrete Beams.—(University of Illinois, Urbana). Study of behavior under load and efficiency of various types of reinforcement against diagonal tension. Tests of simple and restrained beams from 6 to 18 ft. in length. Measurement of deformations in reinforcement by use of strain gages.

Reduced Structural Sections Restored by Means of Concrete.—(Eng. Exp. Station, Ohio State University, Columbus). Study of reinforcing value to attach to concrete or reinforced-concrete beam built around a steel structural member; full-size specimens used.

Badly corroded and scrapped plate girder stringers and I-beams as well as some new Bethlehem I-beams and H-columns were covered with gunite or with poured concrete. In some cases steel reinforcement was added. Specimens tested to failure and observations made of fiber stress deformation and total deflection.

Bond.

Bond between Concrete and Steel.—(University of Akron, Akron, Ohio). Steel rod placed inside piece of wrought iron pipe of fairly large diameter and filled with concrete to within ½ in. of top. Rod then pulled out in usual manner except that compression from machine comes upon end of pipe instead of upon concrete. This does not put concrete in compression as is the usual case.

Chimneys.

Tests of a Reinforced-Concrete Chimney.—(American Concrete Institute committee S-1 on Reinforced-Concrete Chimneys). Tests are being continued on 309-ft. reinforced-concrete chimney of Universal Portland Cement Co. at Duluth under direction of committee S-1 of the American Concrete Institute to secure information on behavior of reinforced-concrete chimneys in service and make a study of factors to be considered in chimney design.

Tests include (a) wind pressures and wind velocities; (b) temperatures; (c) stresses in concrete and steel due to wind load, dead load, shrinkage, time-yield and temperature; and (d) deflections due to wind. All of these items have been given attention during past year. Readings appear to be quite satisfactory except in case of deflection readings, where apparatus has not given accuracy that was expected of it. Readings have been carried on for period of 1 year or more and will be continued.

A report of temperature, wind and strain gage results is given in the report of committee S-1 on Reinforced-Concrete Chimneys.

Slabs.

Experimental Mine Stoppings.—(U. S. Bureau of Standards, Washington). Study of relative effect of static loading and explosive forces on plain and reinforced-concrete slabs from portland and high alumina cement.

Tests of Reinforced-Concrete Slabs.—(University of North Dakota, Grand Forks). Continuing study of relation between width of slab and load which will cause failure in shear (diagonal tension).

V.—SUGGESTED RESEARCHES ON CONCRETE AND RELATED SUBJECTS.

Cement.

Possibility of improving the quality of cement as at present manufactured without greatly increasing cost, especially the benefit to be derived from higher burning temperature and finer grinding.

Development of a better soundness test for cement.

Methods for determining the relative value of various cements as regards strength, permeability and resistance to weathering of concrete.

Cause and prevention of quick-stiffening of cement when mixed with water.

Comprehensive tests of lumnite cement.

Possibilities of lumnite for cold weather construction.

Causes and means of prevention of surface dusting of high alumina cement concrete.

Modulus of elasticity of high alumina cement concrete.

Use of high alumina cement concrete in patching portland cement concrete pavements.

Use of high alumina cement in sand and gravel tests.

Aggregates.

Correlation of tests of physical characteristics of aggregates with service in concrete.

Development of an accurate method for determining durability of aggregate.

Methods for testing the quality of sands for use in concrete—strength and resistance to weathering and abrasion, etc.

Resistance of aggregates to weather and to high temperatures.

Fire resistance of different aggregates.

Significance of the various accelerated soundness tests for aggregates.

Durability of mineral aggregates in general when they are incorporated in concrete subjected to every type of strain or condition that may occur in the use of concrete.

Effect of finely divided materials, such as clay, in aggregate.

Effect of grading of coarse aggregate on strength and yield of concrete.

Effect of grading and quality of fine aggregate on strength of concrete.

Economics of concrete proportioning, that is, relative yield as affected by type of gradation of aggregate and methods of measurement.

Durability of concrete as affected by type of aggregate.

Plain Concrete.

Study of effects of admixtures, integral waterproofing materials and methods of placing on physical constants and structure of concrete.

Effect of fine fillers, such as inorganic silt or clays, diatomaceous earth, etc., on strength, permeability, workability, yield, and resistance to weathering of concrete.

Relative effect of chemical admixtures in concrete, both as curing agents and accelerators.

Relative effects of various proposed methods of curing concrete pavements.

Resistance of concrete to sea water and alkali action.

Expansion and contraction of concrete.

Study of repeated loads and impact stresses.

Fatigue of concrete under repeated static and impact loads.

Accelerated weathering tests for concrete.

Causes of failure of concrete after being in place for long periods.

Effects of freezing and thawing on concrete from various mixes and types of specimen.

Relation, if any, of the percentage of absorption and the permeability of concrete to its resistance to freezing and thawing.

Study of methods of making test specimens with view to more nearly simulating field conditions.

Effect of fineness of cement, curing conditions and time of setting on imperviousness.

Influence of the water-cement ratio on the permeability and absorption of concrete.

Prehydration of cement.

Study of relation between compressive, tensile and flexural strength of concrete.

Effect of the shape and surface conditions of aggregate particles on the tensile and compressive strength of concrete.

Study of correlation between time of setting of cement and rate of increase in concrete strength.

Tensile strength of concrete.

Effect of size and shape of specimen and method of testing on transverse strength of concrete.

Standardization of methods of making flexure tests.

Study of variation in strength of concrete, at mixer, at end of chute, and in place.

Cause and prevention of hair-checks or shrinkage cracks in surface of concrete pavement.

Agencies affecting shrinkage of concrete.

Study of stresses due to moisture and temperature.

Simple modern specifications for concrete materials and concrete based upon strength and workability of the concrete.

Variations in strength with age (100-year program).

Volume changes in concrete.

Waterproofing of concrete.

A positive simple method of measuring workability of concrete, differentiating between unworkability due to lack of water and that due to improper proportioning of aggregates.

Reinforced Concrete.

Effect of using corrosion-resistant steel in lieu of ordinary steel for structures exposed to the action of sea water.

Continued investigations of effect of penetration of moisture on reinforcement, especially when exposed to sea air and sea moisture.

Tests on rerolled rail reinforcing bars.

Torsional resistance of beams.

Bond tests of Lumnite cement concrete.

Effect of shrinkage and flow on strength of reinforced-concrete columns.

Effect of length on strength of concrete columns.

Tests of eccentrically loaded columns.

Tests of rigid frames.

Best methods of preventing the rusting of reinforcing steel resulting in the spalling off of thin coverings of concrete, as happens in concrete lighting standards and poles, etc. Use of Cal, galvanizing, cement paints and other coatings of the steel or surface treatments of the poles; better methods of manufacture, dangers of accelerators in this connection.

Distribution of concentrated loads on reinforced-concrete slabs.

Resistance of one- and two-way slabs.

Effect of repeated and continued loads at high stresses (1) on bond, and (2) on direct compression.

VI—REFERENCES TO PAPERS AND REPORTS ON RESEARCHES PUBLISHED DURING 1926

Cement.

Theory of Hardening of Hydraulic Cements, by Baykoff.

Comptes Rendus, v. 189, p. 128, 1926.

Constitution of Portland Cement, by E. Jänecke.

Jl. Elektrochemie, v. 32, p. 354, 1926.

Stahl und Eisen, v. 24, 1926.

Lime-Silica Index as Measure of Cement Quality, by T. Merriman.

Eng. News-Rec., v. 96, p. 648, April 22, 1926.

Rock Products, v. 29, p. 51, May 29, 1926.

Proposal to grade portland cements in terms of both strength and relative content of cementing constituents; specifications for tensile strength and quality proposed.

Effect of Silica Content on Soundness of Hardened Portland Cement, by F. Kubik.

Toning. Ztg., March 3, 1926.

Digest of Literature on Constitution of Portland Cement Clinker, by R. H. Bogue.
Paper 3, P. C. A. Fellowship, U. S. Bureau of Standards, 1926.
Concrete (CMS), v. 29, July, 1926 to Jan., 1927.

Portland Cement Research, by R. H. Bogue.
Paper 4, P. C. A. Fellowship, U. S. Bureau of Standards, 1926.
Proc. Am. Soc. Testing Mat., v. 26, 1926.

Studies on System Calcium Oxide-Ferric Oxide-Silica, by Hansen and Bogue.
Paper 1, P. C. A. Fellowship, U. S. Bureau of Standards, 1926.
Jl., Am. Chemical Soc., v. 42, p. 1261, March, 1926.

Determination of Free Lime in Hydraulic Cement.
Tech. Paper 4, Dept. of Scientific and Ind. Research, London, 1926.

Determination of Uncombined Lime in Portland Cement, by Bogue and Lerch.
Paper 2, P. C. A. Fellowship, U. S. Bureau of Standards, 1926.
Ind. and Eng. Chem., v. 18, p. 739, July, 1926.

Portland Cement, High Sulphur Content, by F. C. Lea.
Canadian Eng., v. 51, p. 133, July 13, 1926.
Experiments on 5 types of specimens on effect of sulphur on portland cement.

Effect of Calcium Chloride on Cement, by C. E. Platzman.
Rock Products, v. 29, p. 78, Feb. 6, 1926.
Results of European experiments.

High-Alumina Hydraulic Cements, by P. H. Bates.
Rock Produots, v. 29, p. 58, June 26, 1926.

Ferrous and Aluminous Cements, and Considerations on Hydraulic Compounds, by E. Martin.
Moniteur Scientific, v. 26, 1926.

Contribution to Knowledge of Alumina Cement, by H. Eisenbeck.
Chem. Ztg., March 6, 20; April 3, 7, 1926.

Quick Hardening Cements, by F. C. Lang.
Proc. 5th Annual Meeting Highway Res. Board, Part I, p. 145, 1926.

Report of Special Sub-Committee on Survey of Tension Tests of Portland Cement for 1925.
Proc. Am. Soc. Testing Mat., v. 26, 1926.

Correlation Study of Tensile Strengths of Portland Cement Briquets, by Gowen, Leavitt, and Evans.
Bull. 14, Maine Tech. Exp. Sta., April, 1926.

Prediction of 28-Day Tensile Strength of Portland Cements from 7-Day Strengths, by Gowen and Leavitt.
Bull. 15, Maine Tech. Exp. Sta., May, 1926.

Device for Measuring Pressure Used in Molding Cement Mortar Briquets, by F. H. Jackson.
Public Roads, v. 7, p. 104, July, 1926.

Setting Time of Cement Indicated by Machine Operation, by A. A. Jakkula.
Eng. News-Rec., v. 97, p. 66, July 8, 1926.

Concrete.

Report on Use of Accelerators in Portland Cement Concrete, by R. W. Crum.
Proc. 5th Annual Meeting Highway Research Board, Part I, p. 132, 1926.

Effect of Calcium Chloride on Tranverse Strength of Concrete Aged at Various Temperatures, by L. C. Stewart.
Proc. Am. Soc. Testing Mat., v. 26, 1926.

Tests Show How Calcium Chloride Affects Concrete, by A. S. Levens.
Eng. News-Rec., v. 97, p. 214, Aug. 5, 1926.
Admixtures up to 5 per cent increase strength of concrete slightly, but also increase shrinkage.

California Quick-Hardening Concrete.
Rock Products, v. 29, p. 46, March 6, 1926.
Highway Research Dept. finds mixtures that will allow roads opened in 2 or 3 days.

Cement-Lime Mortars, by H. V. Johnson.
Tech. Paper 308, U. S. Bureau of Standards, 1926.

Effect of Lime on Concrete Products, by P. C. Cunnick.
Proc. Am. Concrete Inst., v. 22, p. 239, 1926.

Use of Lime in Concrete Roads, by C. R. Stokes.
Rock Products, v. 29, p. 68, Sept. 4, 1926.

Soaps as Integral Waterproofings for Concrete, by White and Bateman.
Proc. Am. Concrete Inst., v. 22, p. 535, 1926.

Strengthening and Indurating Concrete with Sulphur, by W. H. Kobbe.
Eng. News-Rec., v. 96, p. 940, June 10, 1926.

Role of Aggregate in Concrete, by Goslich.
Zement, p. 308, April 29, 1926.

Analysis of Continuous Concrete Arch Systems, by C. S. Whitney.
Proc. Am. Soc. Civil Eng., v. 52, May, 1926.
Method for analyzing system composed of 2 arch spans with elastic pier.

Stresses in Thick Arches of Dams, by B. F. Jakobsen.
Proc. Am. Soc. Civil Eng., v. 52, Feb., 1926.

Autogenous Healing of Concretes and Mortars, by H. J. Gilkey.
Proc. Am. Soc. Testing Mat., v. 26, 1926.

Seven Years of Experience with Job Control of Quality Concrete, by R. B. Young.
Proc. Am. Concrete Inst., v. 22, p. 79, 1926.

Producing Concrete of Uniform Quality, by R. B. Young.
Proc. Am. Soc. Civil Eng., v. 51, Sept., 1926.
Canadian Eng., v. 51, p. 309, Sept. 21, 1926.

Control of Mixture and Testing of Wilson Dam Concrete, by J. W. Hall.
Proc. Am. Concrete Inst., v. 22, p. 488, 1926.

New Experiences in Concrete Control, by J. G. Ahlers.
Proc. Am. Concrete Inst., v. 22, p. 159, 1926.

Control of Concrete Mixtures on University of Pittsburgh Stadium, by W. S. Hindman.
Proc. Am. Concrete Inst., v. 22, p. 110, 1926.

Control Tests on Construction Jobs in Austria.
Beton und Eisen, No. 4, 1926.

Use of Water-Cement Ratio Specification on Portland Cement Association Building, by McMillan and Walker.
Proc. Am. Concrete Inst., v. 22, p. 122, 1926.

Water-Cement Ratio Specification for Concrete, by McMillan and Walker.
Proc. Am. Soc. Civil Eng., v. 51, Sept., 1926.

Effect of Manipulation on Strength of Field Concrete, by G. W. Hutchinson.
Eng. News-Rec., v. 96, p. 246, Feb. 11, 1926.
Concrete varies according to amount of work given placing.

Corrosion of Concrete, by J. R. Baylis.
Proc. Am. Soc. Civil Eng., v. 51, April, 1926.
Portland cement does not form compounds insoluble in water corrosive to calcium carbonate. Solubility of calcium carbonate depends on alkalinity, hydrogen-ion concentration, and other salts in water.

Durability of Cement Drain Tile and Concrete in Alkali Soils; 4th Progress Report (1923), by Williams and Furlong.
Tech. Paper 307, U. S. Bureau of Standards, 1926.

Protection of Concrete Against Alkali, by E. C. E. Lord.
Public Roads, v. 7, p. 251, Jan., 1926.
Tests by Bureau of Public Roads on treatment of Concrete with tar and paraffin.

Recent Studies on Effect of Alkali on Concrete, by C. J. Mackenzie.
Eng. News-Rec., v. 95, p. 920, Dec. 3, 1925.

Sewer Lining Experiments for Los Angeles County, by A. M. Rawn.
Eng. News-Rec., v. 96, p. 18, Jan. 7, 1926.
Bypass composed of various materials, joints, linings and coatings subjected to flow of septic sewage.

Deterioration of Concrete in Sea Water and Preventive Methods, by G. F. Nicholson.

Paper, 1926 Annual Convention of Pacific Coast Assn. of Port Authorities, Portland, Ore.

Tests on Concrete Piles in Sea Water Undertaken by Navy.

Eng. News-Rec., v. 97, p. 372, Sept. 2, 1926.

Methods of making and suspending 35 test piles under pier at Portsmouth Navy Yard; examination at 6-mo. intervals.

Studies of Curing Concrete in Semi-Arid Climate, by C. L. McKesson.

Eng. News-Rec., v. 96, p. 452, March 18, 1926.

Use of substitutes for water curing justified where no water can be had; 7-day water curing sufficient.

Finishing and Curing of Concrete Roads, by C. L. McKesson.

Proc. Am. Soc. Civil Eng., v. 51, Aug., 1926.

Effect of Varied Curing Conditions Upon Compressive Strength of Mortars and Concretes, by H. J. Gilkey.

Proc. Am. Concrete Inst., v. 22, p. 395, 1926.

Effect of Hot Water Vapor on Concrete Products, by A. L. Miller.

Rock Products, v. 29, p. 86, Sept. 4, 1926.

Results of tests presented at Convention of Northwest Concrete Products Assn. Tests to discover effect of application of water vapor at between 125° and 212° F. on strength of mixture of portland cement, sand and water during first 8 hr. after mixing.

Extensibility of Concrete, by W. K. Hatt.

Proc. Am. Concrete Inst., v. 22, p. 364, 1926.

How to Prevent and Restore Frozen Concrete, by A. M. Bouillon.

Eng. News-Rec., v. 96, p. 408, March 11, 1926.

Effect of frost upon freshly-made concrete; suggestions on discovery and restoration of frozen concrete.

Stresses in Concrete Pavements Computed by Theoretical Analysis, by H. M. Westergaard.

Public Roads, v. 7, p. 25, April, 1926.

Proc. 5th Annual Meeting Highway Research Board, Part I, p. 30, 1926.

Concrete Pavement Design, by Teller and Pauls.

Proc. Am. Concrete Inst., v. 22, p. 314, 1926.

Efficiency in Supervision of Construction of Concrete Road Surfacing, by J. L. Harrison.

Proc. Am. Concrete Inst., v. 22, p. 279, 1926.

Tests of Vibrolithic Concrete, by Division of Tests, U. S. Bureau of Public Roads, by Teller and Proudley.

Public Roads, v. 7, p. 36, April, 1926.

28-day tests on concrete slabs constructed by ordinary methods and Vibrolithic process to obtain data on relative strength.

Report on A Field Experiment on Introduction of Planes of Weakness in Concrete Slabs, by W. D. Somervell.

Proc. 5th Annual Meeting Highway Res. Board, Part I, p. 122, 1926.

Summary of Tension Tests of Concrete Briquets Reinforced with Steel Fabric, by H. E. Breed.

Proc. 5th Annual Meeting Highway Res. Board, Part I, p. 118, 1926.

Report on Rio Vista, Calif., Subgrade Treatment Experiments, by C. L. McKesson.

Proc. 5th Annual Meeting Highway Research Board, Part I, p. 123, 1926.

Permeability of Portland Cement Concrete, by W. H. Glanville.

Tech. Paper 3, Dept. of Scientific and Industrial Research, London, 1926.

Tests of Cement-Asbestos Pipe, by W. S. Pardoe.

Eng. News-Rec., v. 96, p. 169, Jan. 28, 1926.

Results of tests on 150-mm. pipe of 15 mm. thickness using 85 per cent portland cement and 15 per cent asbestos fibre.

Proportioning Concrete, by W. Vieser.

Zement, Sept. 2 to Oct. 14, 1926.

Concrete Strength Made Uniform by Careful Proportioning, by Z. Witkin.

Eng. News-Rec., v. 97, p. 258, Aug. 12, 1926.

Comparisons in California building between ordinary volumetric measurement, weight measurement and inundation method.

Vary Mix Design for Concrete to be Used at Different Ages, by R. T. Giles.

Eng. News-Rec., v. 97, p. 510, Sept. 23, 1926.

Transmission and Absorption of Sound by Some Building Materials, by Eckhardt and Chrisler.

Scient. Paper 526, Nat. Bureau of Standards, 1926.

Concrete of Specified Strength, by O. Graf.

Reprint, *Zeits. Ver. Deutscher Ing.*, No. 12, p. 411, 1926.

Relation of 7-Day to 28-Day Compressive Strength of Mortar and Concrete, by W. A. Slater.

Proc. Am. Concrete Inst., v. 22, p. 437, 1926.

Strength of Mortar and Concrete as Influenced by Grading of Sand, by J. G. Rose.

Public Roads, v. 7, p. 106, July, 1926; (Discussion by T. C. Powers, *Public Roads*, v. 7, p. 174, Oct., 1926).

Relation between grading of sand for use in concrete and strength developed in mortar and concrete has led to development of graph which may be used as basis for preliminary judgment of quality of sands proposed for use.

Concrete in Tension, by A. N. Johnson.

Proc. Am. Soc. Testing Mat., v. 26, 1926.

Public Roads, v. 7, p. 90, June, 1926.

Comparison of Transverse and Compressive Tests of Concrete, by H. F. Clemmer.
Public Roads, v. 7, p. 67, May, 1926.
Discusses whether compression test may be taken as direct measure of transverse strength of pavement. Laboratory tests to determine relation between flexural and compressive strength of same concrete.

Transverse Testing of Concrete, by Clemmer and Burggraf.
Proc. Am. Concrete Inst., v. 22, p. 304, 1926.

Static and Impact Strains in Concrete, by J. T. Thompson.
Public Roads, v. 7, p. 93, July, 1926.
Comparison of moduli of rupture of beams under static and impact loads.

Tests of Concrete Staves; Report of Committee P-4 on Concrete Staves.
Proc. Am. Concrete Inst., v. 22, p. 662, 1926.

Properties and Manufacture of Concrete Building Units.
Circular 304, U. S. Bureau of Standards, 1926.

What are Most Significant Tests for Concrete? by A. T. Goldbeck.
Proc. Am. Concrete Inst., v. 22, p. 386, 1926.

Device for Eliminating Capping of Concrete Cylinders, by W. F. Purrington.
Proc. Am. Soc. Testing Mat., v. 26, 1926.

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Tech. Paper 5, Dept. of Scient. and Ind. Research, London, 1926.

Sieve Analysis, Slump and Flow Tests and Their Application, by O. Graf.
Beton und Eisen, June 20, 1926.

Reinforced Concrete.

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Canadian Eng., v. 50, p. 601 and 636, May 25 and June 8, 1926.

Shear Tests of Reinforced-Concrete Beams, by Slater, Lord, and Zipprodt.
Tech. Paper 314, Nat. Bureau of Standards, 1926.

Steel I-Beams Haunched with Concrete, by H. M. MacKay.
Canadian Eng., v. 51, p. 671, Nov. 30, 1926.
Tests on panels and other units showing additional strength afforded by concrete in ordinary haunched beam and slab construction—Paper before Am. Inst. of Steel Construction.

"J. and L." Junior Beam Construction Tested, by C. R. Young.
Canadian Eng., v. 51, p. 255, Aug. 31, 1926.
Test panel carries normal dead load and 4.38 times rated live load without failure; deflections for twice rated live load much less than those permissible for plastered ceilings; hanger tests satisfactory.

Haunching as Reinforcement for Steel Beams, by C. R. Young.
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Combined Concrete and Timber in Flexure, by G. D. Burr.
Bull. 37, Univ. of Wash. Eng. Exp. Sta., 1926.

Outline of Tests on 300-ft. Reinforced-Concrete Chimney, by B. Wilk.
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Stresses in Helically Reinforced-Concrete Columns, by Zesiger and Affeldt.
Proc. Am. Soc. Civil Eng., v. 52, Jan., 1926.

Analyzes effect of helical reinforcement on concrete core of helically reinforced-concrete column, and develops formulas for stresses which obtain in steel and concrete with varying percentages of reinforcement.

Stress Measurements in Concrete Pavements, by L. W. Teller.
Proc. 5th Annual Meeting Highway Research Board, Part I, p. 67, 1926.

Report of Investigation of Economic Value of Reinforcement in Concrete Roads, by C. A. Hogentogler.
Proc. 5th Annual Meeting Highway Research Board, Part II, p. 8, 1926.

Effect of Lightning on Reinforced-Concrete Pavement, by W. E. Wheat.
Proc. Am. Soc. Civil Eng., v. 52, Sept., 1926.

Formulas for Design of Rectangular Floor Slabs and Supporting Girders, by H. M. Westergaard.
Proc. Am. Concrete Inst., v. 22, p. 26, 1926.

Distribution of Reinforcing Steel in Concrete Beams and Slabs, by B. S. Myers.
Proc. Am. Soc. Civil Eng., v. 52, Aug., 1926.

To determine correct amount of reinforcing steel required in top of concrete beams and slabs at supports for varying degrees of continuity.

Effective Width of Concrete Bridge Slabs Supporting Concentrated Loads, by E. F. Kelley.
Public Roads, v. 7, p. 7, March, 1926.

Results of three investigations on stress distribution made by Univ. of Ill., State Highway Dept. of Ohio, and U. S. Bureau of Public Roads.

Slabs for Delaware River Bridge Tested.
Public Roads, v. 7, p. 68, May, 1926.

Tests to check design in slabs for bridge and method of support.

Reinforced-Concrete Pipe Tests, by A. E. Warren.
Canadian Eng., v. 51, p. 346, Oct. 5, 1926.

Charts for Reinforced-Concrete Poles, by Gillespie and Wilson.
Canadian Eng., v. 51, p. 175, Aug. 3, 1926.

Graphical methods for determining flexural resistance, deflection, etc., of reinforced-concrete poles of typical section.

Report of Committee E-5 on Aggregates.

After several futile attempts to formulate specification limits for some characteristics of aggregates. Committee E-5 realized the necessity of having for its guidance, authoritative statements concerning the present state of knowledge of the various classes of such materials. Sub-committees were therefore appointed to prepare authoritative and exhaustive reports upon the various factors in fine aggregate, gravel, crushed stone and slag that affect strength, durability, density and other properties of concrete. The reports on gravel and crushed stone have been completed and are presented herewith. A progress report on slag is also presented.

These reports indicate that some specification limits can be set from present knowledge, and that further research is needed on many points now in doubt.

It is the hope of the committee that the valuable information assembled in these reports will be interesting and useful to the members as well as to the committee itself.

Requirements of Gravel as an Aggregate for Concrete.

Report of Sub-Committee on Gravel of Committee E-5.

Introduction.

This sub-committee has been assigned the duty of preparing an "authoritative and exhaustive report upon the various factors in gravel aggregates that affect the strength, durability, density, and other properties of concrete for various uses." The assignment includes consideration of characteristics due to natural causes, methods of preparation, and methods of use.

The preparation of a report for presentation to the committee before the 1927 convention, which could be considered "authoritative and exhaustive," seemed impractical, in view of the limited time available and the apparent lack of comprehensive information on many of the questions involved. In this report, therefore, we have attempted only to discuss briefly certain of the more important factors, summarizing some of the most complete information on them and pointing out wherein further information is most needed.

Gravel consists of particles, coarser than about ¼ in., resulting from the natural disintegration of rock. That gravel is an important structural material worthy of careful consideration is illustrated by the fact that there are a considerable number of established companies producing over 1,000,000 tons annually, and that the annual production of sand and gravel for all purposes in the United States is more than 175,000,000 tons.

Exhaustive researches in concrete aggregates have been carried out during the past 10 to 15 years in many different laboratories. Most of these studies, however, have been confined to the effect of variations in grading and proportioning of aggregates on the strength and density of concrete; comparatively little has been done which gives conclusive information on what constitutes minimum requirements so far as quality of particles is concerned. These tests have shown that the strength and other properties of concrete are dependent to a large extent on the relative quantities of cement and water in the mix, so long as the aggregate is structurally sound, durable, and clean. The grading of the aggregate, particularly its uniformity of grading, is, however, important from the viewpoint of economy and of the uniformity in quality of the resulting concrete.

Most of our present-day specifications for concrete aggregates are "rule of thumb" documents based on engineering judgment of local conditions. It is essential, of course, that the specification of requirements for gravel, as for any natural material, take into consideration the characteristics of deposits in the different localities; this is particularly true of grading. However, certain fundamental requirements are applicable to all localities

and should be dictated by the nature of the service to which the materials will be subjected. Practical considerations demand that aggregates have such physical characteristics that definite proportions may be specified with the assurance that concrete of the required quality will be obtained. From this point of view it is essential that suitable requirements as to hardness and strength, durability, cleanness, and grading, particularly uniformity of grading, be stated. The statement of requirements for quality of aggregate particles is complicated by the lack of significance of the chemical composition of the aggregate, and must depend entirely on physical characteristics. The usual specification attempts to do this by specifying that the gravel shall consist of particles which are "hard, strong, durable, uncoated, and free from injurious amounts of soft, friable, thin, elongated, or laminated pieces, alkali, organic, or other deleterious matter." In addition to this formidable catalogue, a requirement for grading is invariably included.

Lack of information concerning the effect of many of these characteristics has made it impossible to place proper limitations on all of the different factors involved. Grading is the only factor of which it may be said that anything approximating adequate information has been obtained; only the most limited data are available on what constitutes proper limitations for characteristics, such as hardness, strength, durability, etc. The principal part of this report is devoted to discussions of the physical characteristics of the aggregate particles.

HARDNESS AND STRENGTH OF GRAVEL.

What constitutes sufficient hardness and strength in gravel for concrete for different purposes? Probably, but not positively, the service to which the concrete is to be subjected, whether to resist compressive and tensile stresses, abrasion, impact, weather, etc., has an important bearing on what the requirements for these factors should be. The studies of this subcommittee indicate a woeful lack of information on the question—entirely insufficient to enable it to make definite recommendations for specification limits.

A number of researches have been carried out by different laboratories in attempts to establish relationships between some measure of the hardness and strength of gravel and its concrete making properties. Most of these studies have looked toward the development of some modification of the American Society for Testing Materials' Standard Deval Abrasion Test for crushed stone. An impact test for gravel,[1] in which individual particles are subjected to a blow from a falling steel ball, has been studied by the Bureau of Public Roads, the New Hampshire State Highway Department, and by others. A pressure test on individual gravel particles, made with an instrument known as the Douglas machine, has been studied by the

[1] See 1922 *Proc.* Am. Soc. for Testing Mat., Part II, "An Impact Test for Gravel," by F. H. Jackson, with discussions by W. F. Purrington and Duff A. Abrams.

Iowa State Highway Commission. Abrams, at the Structurals Materials Research Laboratory (now Research Laboratories of the Portland Cement Association), made studies of a crushing test,[1] in which a confined sample of graded gravel was loaded in a testing machine and the effect measured by the differences in sieve analyses made before and after the test.

The modification of the Deval Abrasion Test, to which most attention has been given, is that originally proposed by A. S. Rea of the Ohio State Highway Department and described in United States Department of Agriculture Bulletin No. 1216. This method consists of testing a graded sample of gravel in the Standard Deval cylinder, with cast iron shot included in the sample as an abrasive. (Test method referred to in more detail below.)

State Highway Departments, feeling the need of a tangible measure of the hardness and strength of gravel particles, have in many cases specified a limiting percentage of wear, as measured by this method. In general, the limitation is placed at 15 per cent, although in a few cases as low as 10 and as high as 25 per cent is specified.

This test, or some modification of it, seems to offer possibilities for measuring the hardness of gravel and for comparing its hardness with that of crushed stone, measured by the Standard Deval Abrasion Test. However, data obtained up to this time have not established a definite relationship, nor, so far as we have been able to learn, do they indicate any dependence of the concrete making properties of gravel on its hardness as measured by this test. It should be pointed out that the results of any modification of the Deval test depend greatly on the size and shape of the particle, and that the method suggested by Mr. Rea does not permit of making the test on gravels which do not contain the coarser sizes of particles.

The impact test proposed by Jackson[1] is not properly, nor is it claimed by Mr. Jackson to be, a method for measuring the hardness of gravel as a whole. It seems, however, that it may provide a basis for the elimination of gravels with too high a percentage of soft and friable particles. However, sufficient information has not been obtained on this test to draw definite conclusions concerning its practicability. The method is, fundamentally, only a refinement of the "rule of thumb" test of striking particles of gravel with a hammer and observing their hardness. Mr. Jackson states that, as a result of his more recent studies of this method, he has come to the conclusion that the procedure outlined in his paper is somewhat too severe, and he plans to make a number of changes in the details of the test method.

The pressure test with the Douglas machine, used by the Iowa State Highway Commission, is for the purpose of determining the percentage of "rotten stone" in a sample of gravel. A 1000-gram sample is subjected to

[1] See 1922 *Proc.* Am. Soc. for Testing Mat., Part II, "An Impact Test for Gravel," by F. H. Jackson, with discussions by W. F. Purrington and Duff A. Abrams.

a visual examination and the particles classified as hard, soft, or partially disintegrated (rotten stone). In case of doubt as to whether a particle should be classified as soft or partially disintegrated, it is subjected to a steady pressure in the machine, as follows:

Particles retained on 0.742-in. sieve	75 lb.
Particles passing 0.742-in. sieve and retained on 0.371-in. sieve	60 lb.
Particles passing 0.371-in. sieve	35 lb.

Particles which fail in the test are classified as "rotten stone." Mr. Crum states that this method of test has been found to be very satisfactory.

The crushing test described by Abrams seems to offer promise as a method of measuring the strength of gravel particles, although, as pointed out below, studies to date have not indicated any relation between this test and the concrete making properties of gravel. It was adapted from a method used in Austria.[2]

The time available for the preparation of this report did not permit of anything approximating a detailed survey of the existing information on this question. Studies have been made of typical data, which are probably as comprehensive as any, from Jackson and Pauls' paper on "Accelerated Wear Tests on Concrete Pavements"[3] and from unpublished data (Series 135) of the Structural Materials Research Laboratory.

These investigations included tests of a wide variety of gravels and other aggregates. Compression, transverse, and abrasion tests of concrete and results of various modifications of the Deval Abrasion Test, specific gravity, absorption, and crushing test of the aggregates were made. A summary of the data of the tests with gravel is given in Appendices 1 and 2 of this report.

It seems obvious that these data do not provide sufficient information to serve as a basis for fixing limitations on the different characteristics measured. The conclusion which seems to be logical from a study of them is that, within the range of these aggregates and for the conditions of these tests, no well defined relation exists between the results of the Deval Abrasion Test, the crushing test, specific gravity, absorption, or unit weight, and the concrete making properties of a gravel as measured by the compressive strength, transverse strength, or resistance to abrasion in the Talbot-Jones Rattler Test. As stated above, the gravels tested represented a wide range in quality and were from widely separated localities.

In view of the lack of definite information showing any dependence of concrete-making properties of a gravel on its hardness and strength, as measured by any of the methods studied, this sub-committee does not feel that it is in a position to offer definite recommendations for specifying

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[2] See *Proc.* Int. Assn. Testing Mats. (1912), 2d Section, Paper XXVI.

[3] See 1924 *Proc.* Am. Soc. for Testing Mat., Part II, and *Public Roads*, Vol. 5, No. 3, May, 1924.

hardness and strength of gravel which could be considered as on a sound basis of knowledge or as an improvement over existing specifications.

It does not seem unreasonable to suppose that, *except for soft, friable, and partially disintegrated particles and particles which are not durable on exposure to the weather,* any material which has withstood the action of the elements necessary for the formation of gravel consists of particles hard enough for the usual service in concrete. We feel, however, that a more complete study of existing information and further comprehensive investigations are needed, definitely directed toward a solution of this problem.

DURABILITY OF GRAVEL.

Considerable work has been done in an attempt to develop a test which may be used to determine whether or not an aggregate will be durable in concrete. The method of test for soundness of aggregate which has received most attention is that recommended by the American Association of State Highway Officials and described in United States Department of Agriculture Bulletin No. 1216. This test consists of alternately soaking a sample of the aggregate in a saturated solution of sodium sulphate (Na_2SO_4) for twenty hours and drying it in an oven for four hours. This operation is repeated five times. This test seems to have been developed by M. Brard in connection with studies of the resistance of building stone to frost action. It is described in the 1828 *Annales de Chemie et de Physique*, v. 38, p. 160, and is referred to in an appendix by Smith in Vicat's Treatise on "Calcareous Mortars and Cements" (translated from the French by J. T. Smith-J. Weale, London, 1837). The test has been used by various investigators in studies of building stones, clay products, concrete, and mortar since that time. Studies of this method of test are being carried out at the U. S. Bureau of Public Roads, U. S. Bureau of Standards, Allegheny County Highway Laboratory, research laboratories of the Portland Cement Association, Minnesota State Highway Department, and others. Most of the work, however, has been on crushed stone; only a few studies have been made on gravel. The general conclusion thus far seems to be that this test method is not a necessarily true indication of the durability of an aggregate in concrete; many aggregates which fail in this test apparently give satisfactory results in concrete.

Shale is the most common material, which sometimes occurs in gravel deposits, known to be unsound in concrete. Its occurrence, however, is not general and is limited to a comparatively few localities. Comprehensive studies of the effect of shale in Minnesota gravels have been carried out by F. C. Lang of the Minnesota State Highway Department. The results of these tests are discussed in Appendix 3 of this report. They show that even small percentages of shale cause considerable reductions in compressive strength.

Mr. Lang points out that quantities of shale less than will produce a harmful effect on the compressive strength should not be permitted in

gravel, particularly for wearing surfaces, as he feels that the shale will invariably be worked to the surface and form spots which will disintegrate at an early period under the action of the weather and traffic. He points out that their experience has caused them to limit the amount of shale permissible in gravel for concrete highway construction to 0.5 per cent by weight.

Most of the state highway departments place only the time-honored "free from" limitation on shale and similar materials, although Kansas, Iowa, and Minnesota specify a maximum of 0.5 per cent shale, while Kentucky places a limitation of 1 per cent on shale, slate, etc., and Michigan 3 per cent on shale and other non-durable particles.

Soft, friable, laminated, and light-weight particles should be taken into account in the consideration of durability of gravels, as well as those particles which are not durable on exposure to the weather. Probably no one test can be devised which will take into account these different factors.

Further studies of accelerated freezing and thawing tests, such as the ones using sodium sulphate or sodium chloride, will undoubtedly result in the development of a method for judging of the resistance of gravel particles to the action of the weather.

This sub-committee believes that any soundness test for gravel should be used to determine the percentage of unsound particles, and that limitations should be stated on this basis rather than to specify that no particles shall exhibit evidence of unsoundness.

Soft, friable, and laminated particles would, undoubtedly, in many cases be attacked by a soundness test. As an additional precaution, it is suggested that an impact test along the lines described by Jackson, or a pressure test similar to that used by the Iowa State Highway Commission can be adapted to the limitation of such particles.

In view of the fact that the most important objection claimed for relatively small quantities of soft and non-durable particles is that, due to their light weight, they are floated to the top and thus form a wearing surface in which pits will occur, this sub-committee suggests a limitation of the amount of such particles on the basis of their specific gravity. A specific gravity of about 2.0 (apparent) is suggested as a practical dividing line, since it will be generally recognized that practically all gravel will have a specific gravity well above this value (2.4 probably the low limit in the vast majority of cases), and that the definitely objectionable particles, which will be floated to the surface of concrete, will generally have a specific gravity lower than 2.0 (about 1.6 in the case of shale). The use of a liquid of high specific gravity, such as zinc chloride ($ZnCl_2$), has been tried with success. A method of making the test, with particular reference to shale, is suggested in U. S. Department of Agriculture Bulletin No. 1216. Tests have also been carried out in which lead acetate was the liquid.

This sub-committee suggests that recommendations for durability of gravel should be for the purpose of eliminating particles of low specific

gravity which will have a tendency to be floated to the surface and particles which will disintegrate in the weather.

Judgment of the durability of a gravel might, therefore, be based on:

(1) The percentage of particles which will not withstand the action of the weather, as measured by a suitable accelerated freezing and thawing test.

(2) The percentage of soft, friable, and laminated particles which will not withstand a suitable impact test or pressure test.

(3) The percentage of light-weight particles which will be floated by a liquid of suitable specific gravity—about 2.0.

It is recognized that there probably would be much overlapping of these three tests; that is, particles eliminated by one of them would also be eliminated by one or both of the others. It is hoped that further work of this sub-committee will result in the development of a suitable procedure for these tests.

CLEANNESS OF GRAVEL.

The usual requirement for clean gravel is that it be free from injurious amounts of coated particles, clay and silt, loam, alkali, and other deleterious substances. In general, no comprehensive investigations have been carried out for the purpose of making a quantitative determination of the effect of these various factors.

The effect of coatings on gravel will obviously vary with the nature and use of the concrete and is difficult of determination in the laboratory.

Clay or silt, provided it is free from organic matter, or other deleterious material, will not react chemically with the cement and may be present in considerable quantities without producing an appreciable effect on the strength of concrete. Nevertheless, clay and silt may be objectionable from a number of considerations. In concrete highway construction it is felt that the presence of clay in the aggregate plays an important part in the production of surface scaling. In reinforced-concrete work, where fairly wet mixtures are used, clay is apt to be worked to the surface and form laitance, thus causing a weak section, particularly at the joining of two days' work. This is particularly objectionable on exposed surfaces. It is obvious from this that a definite measurement of the effect of the various quantities of clay and silt is not easy. Highway engineers have probably given more intelligent consideration to the effect of finely divided particles in concrete aggregates than others, but, in general, they have limited their consideration of them to the fine aggregates. It is common practice to limit the amount of material removed by decantation from sand to 3 per cent, although a number of state highway departments make their limitations as rigid as 2 per cent. If over 3 per cent of finely divided material is objectionable in sand, it would seem that a smaller quantity than 3 per cent would be objectionable in gravel. However, the effect of inert, finely divided particles, such as clay and silt, depends to a great extent on the use to which the concrete is to be put, and this committee feels that it

does not have sufficient information to justify it in suggesting limitations for all uses. Comprehensive studies of the effects of finely divided materials on the strength of concrete are given in Bulletin 8 of the Structural Materials Research Laboratory, "Effect of Hydrated Lime and Other Powdered Admixtures in Concrete," by Duff A. Abrams. However, we feel that tests should be carried out to determine the effects of various amounts of finely divided material, not only on the strength, but particularly on the surface condition of the concrete.

Other deleterious materials, such as loam, organic matter, alkali, etc., should be limited, based on the effect of individual substances on the quality of the resulting concrete.

The quantity of loam and organic matter should be limited by their effect on the strength of the concrete. While it is recognized that, generally, gravels are not apt to contain sufficient organic matter to be harmful to concrete, it would seem desirable to guard against the unusual case by placing a limitation based on the Abrams-Harder Colorimetric Test, adopted as standard for the American Society for Testing Materials. It is suggested that this test should be carried out on material finer than No. 4 mesh sieve and the results calculated in terms of the total aggregate.

The presence of alkalis in gravel is not common. In general, tests have shown them to have little effect on the strength of the concrete for the quantities in which they are apt to be found. They would be objectionable in certain classes of concrete work, in the effect which they would have in producing efflorescence on the surface. Data on the effect of sulphate waters, sodium chloride, and various industrial wastes in concrete are given in Bulletin 12 of the Structural Materials Research Laboratory, "Test of Impure Water for Mixing Concrete," by Duff A. Abrams.

Other "deleterious substances" is at best a somewhat indefinite classification, and it is urged that work be carried out to determine what these substances are. Materials such as coal, lignite, mica, pyrite, oils, etc., should be considered.

GRADING OF GRAVEL.

Considerable information is available on the effect of grading of aggregate, and specifications for grading can be written with a fair degree of knowledge of the effect on the resulting concrete. Such specifications should emphasize uniformity of grading and must take into account, not only the most desirable grading from the viewpoint of the quality of concrete, but also economy as governed by local conditions. For example, the specification of ¼ to 2½-in. gravel as the only size suitable for highway construction would not be economical, when it is considered that in many localities a material meeting this specification could not be produced. A specification for grading of gravel must, therefore, be based on the premise of securing materials which will give the desired results consistent with the greatest economy. Different gradings can be used to obtain concrete of the same quality by making proper variations in the proportions. Too

much emphasis, perhaps, has been placed on the specification of a definite grading and on the use of the coarser-sized particles without giving proper consideration to the most economical grading.

This sub-committee feels that it is not within its province to specify sizes of gravel for different uses, but that these should be determined by local conditions. Specifications for grading should also be predicated on a study of the material in different localities. We recommend the following form of specification as tending to promote uniformity of grading:

Passing (maximum size) inch sieve	95 to 100 per cent	
Passing (½ maximum size) inch sieve	— to —	Spread not more than 35 per cent
Passing No. 4 sieve	0 to 10 per cent	

It will be recognized that each specification for grading will require separate consideration of the proportions which should be used to produce concrete of the quality desired.

RESISTANCE TO HIGH TEMPERATURES.

The mineralogic composition of the aggregate undoubtedly has an important bearing on the resistance of concrete to high temperatures. It is generally recognized, however, that the efficiency of an aggregate in fireproof construction is not necessarily directly proportional to its resistance to high temperatures in the laboratory. The fire hazard, ability of the material to resist the spread of fire, the frequency with which high temperatures occur, and the period of exposure, should be taken into consideration in determining upon the suitability of an aggregate for fireproof construction. This question is not discussed in this report, as Committee E-4 of the American Concrete Institute has in charge studies of the "Fire Resistance of Concrete" and have published the results of their findings in the 1925 *Proceedings* of the American Concrete Institute.

RECOMMENDED STUDIES.

More complete studies and further comprehensive researches are required, definitely directed toward the determination of the relation of the physical characteristics of gravel to its concrete-making properties. It is suggested that particular emphasis should be placed on studies of lean mixtures tested at advanced ages, since in such concrete the quality of the aggregate will be of most importance.

Further studies of hardness and strength of gravel particles should be made, and it is suggested that modifications of the Standard Deval Test and the crushing test are worthy of further attention. In studies of such tests, careful attention should be given to the effect of the grading of the aggregate. In selecting the sizes of materials for a standard test, the wide range in sizes of gravel should be kept in mind. For example, a test which requires the use of 1½-2 in. particles would not be applicable in

certain localities where the largest particles of gravel available may be under 1½ in.

Since the minimum degree of hardness which a crushed stone should have to give good service in concrete pavements seems to be fairly definitely fixed, it would seem desirable to determine the relationship between the Standard Deval Abrasion Test on crushed stone and some modification of the Deval Test for gravel. This might be done by using crushed stone for both tests, making the standard test on the crushed material and the modified test on the same material for which the particles had been rounded in a ball mill. Work of this nature is now under way at the Bureau of Public Roads. Information might also be obtained by making the standard test on crushed gravel and the modified test on rounded particles from the same deposit.

More conclusive information is required in order to make definite determinations of the soundness and durability of gravel. Studies of the accelerated freezing and thawing test should be continued. The impact and pressure tests mentioned above seem to offer promise as a method for determining the amount of soft and friable particles, and further studies should be made of these methods. The determination of light-weight particles by flotation on a liquid of high specific gravity seems to be a worthy subject for further researches.

The effect of finely divided material in gravel, such as clay, silt, loam, etc., should be studied with particular attention to the surface condition of the concrete made from it.

More definite information concerning the occurrence and effect of alkali in gravel is needed before consideration can be given to placing definite limitations on it.

SPECIFICATION FOR GRAVEL.

This sub-committee recommends that a specification for gravel take into consideration the following factors:

1. *Hardness and Strength.*—Probably some modification of the Deval Abrasion Test or of the crushing test can be adapted to the specification of these properties. Further studies are required before definite limitations can be stated.

2. *Durability.*—Particles which are not durable on exposure to the weather and soft, friable, laminated, and light-weight particles should be limited, since they contribute to the lack of durability of gravel. Consideration should be given to the development of test methods along three lines:

(a) An accelerated freezing and thawing test, for the limitation of particles not resistant to the weather.

(b) An impact test such as that proposed by Jackson, or a pressure test such as used by the Iowa State Highway Commission, for the limitation of soft and friable particles.

(c) A separation on the basis of specific gravity, for the limitation of light-weight particles.

3. *Cleanness.*—In considering the cleanness of gravel, cognizance should be taken of clay and silt, coated particles, organic impurities, and other deleterious materials. Clay and silt, and probably loose coatings on gravel, should be limited by a decantation test made on material screened from the gravel through a No. 4 sieve. Organic impurities should be limited by an adaptation of the colorimetric test. Other deleterious materials would be limited to some extent by the decantation test, but further studies should be made to identify such materials and to develop methods for their determination.

4. *Grading.*—The grading of gravel which is most economical depends on local conditions, and this should be recognized in preparing specifications. Any specification for grading should promote the use of a uniform product. The following general requirements are considered desirable:

Passing (maximum size) inch sieve	95 to 100 per cent	
Passing (½ maximum size) inch sieve	— to —	Spread not more than 35 per cent
Passing No. 4 sieve	0 to 10 per cent	

Such a specification requires, of course, that separate consideration be given to the proportions for each different grading.

STANTON WALKER, *Chairman.*
DUFF A. ABRAMS,
JOHN G. AHLERS,
R. W. CRUM.

APPENDIX 1

Summary of Tests on Effect of Quality of Gravel Particles on Their Concrete-making Properties.

(Summary of data from "Accelerated Wear Tests of Concrete Pavements" by Jackson and Pauls.[1])

Typical data showing the effect of quality of gravel particles on their concrete-making properties are given in Jackson and Pauls' paper on "Accelerated Wear Tests of Concrete Pavements." These tests show comparisons between the concrete-making properties of different aggregates and certain physical characteristics.

Fig. 1 from Jackson and Pauls' paper gives data for nine gravels from different sections of the country as follows:

No. 1. Granitic Gravel from Hancock County, Iowa.
No. 2. Quartz Gravel from Richland Country, South Carolina.
No. 3. Quartz Gravel from Potomac River near Washington, D. C.
No. 4. Quartz Gravel from St. Joseph County, Indiana.
No. 5. Dolomite Gravel from Will County, Illinois.
No. 6. Quartz Gravel from Vandenberg County, Indiana.
No. 7. Quartz Gravel from Hampden County, Massachusetts.
No. 8. Limestone Gravel from Warren County, Indiana.
No. 9. Quartz Gravel from Marathon County, Wisconsin.

The specific gravity and absorption of each gravel was determined. A modified Deval Abrasion Test was made for five of the gravels, using the method of test recommended by the American Association of State Highway Officials and described in the United States Department of Agriculture Bulletin No. 1216.[2] Concrete specimens of a 1 : 1½ : 3 mix were made from each gravel and tested for compressive strength, transverse strength, and resistance to abrasion, as measured by the Talbot-Jones Rattler Test. The same sand, a well graded concrete sand from the Potomac River, was used as the fine aggregate in all tests. The gravel was used, graded as received; the grading is not reported in the paper, only the limiting sizes being given.

In Figs. 1 and 2 the gravels have been arranged in order of their compressive strength and the results of the strength tests, Talbot-Jones rattler tests, specific gravity, absorption, and Deval abrasion tests plotted.

[1] See 1924 *Proc.* Am. Soc. for Testing Mat., Part II, and *Public Roads*, Vol. 5, No. 3, May, 1924.

[2] This method consists of running a 5,000-gram sample consisting of 25 per cent of each ½-¾ in., ¾-1 in., 1-1½ in., and 1½-2 in. material in the Standard Deval Abrasion machine for 10,000 revolutions with 6 1⅞ in. cast iron spheres added as an abrasive. The percentage of wear is based on the amount of material finer than 1/16-in. mesh sieve after the test.

It is evident that no definite relation is indicated between the compressive strength and any of the other variables measured. On the contrary, the indications are that, for these tests, the modulus of rupture (transverse strength), depth of wear, absorption, percentage of wear of the aggregates, and specific gravity are entirely independent of the compressive strength.

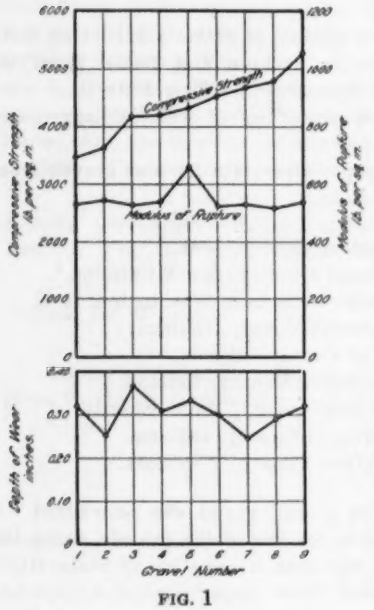

FIG. 1

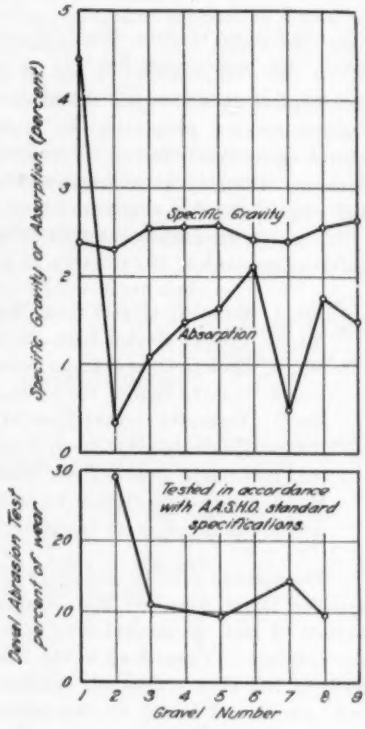

FIG. 2

FIG. 1.—SUMMARY OF CONCRETE TESTS OF GRAVEL.

Data from "Accelerated Wear Tests of Concrete Pavements," by Jackson and Pauls.

FIG. 2.—SUMMARY OF PHYSICAL TESTS OF GRAVEL.

Data from "Accelerated Wear Tests of Concrete Pavements," by Jackson and Pauls.

In the paper Messrs. Jackson and Pauls state the conclusion that the modified abrasion test for gravel in its present form is not an indication of the wear-resisting properties of gravel used in concrete. They suggest that if the severe impact of the steel balls were decreased, more indicative results might be secured.

APPENDIX 2

Summary of Tests on Effect of Quality of Gravel Particles on Their Concrete-making Properties.

(Summary of tests from unpublished data of the Structural Materials Research Laboratory, Series 135, "Tests of Concrete Made from Miscellaneous Coarse Aggregates.")

Tests of twenty-four gravels from unpublished data of the Structural Materials Research Laboratory are included in this appendix through the courtesy of Prof. D. A. Abrams. The data are shown in Figs. 1, 2 and 3 with the values arranged in order of the compressive strength.

For the concrete tests the gravels were separated into three sizes and recombined in the proportions of 25 per cent of No. 4-⅜ in. size, 50 per cent of ⅜ in.-¾ in., and 25 per cent of ¾ in.-1½ in. material. The gravels were made into concrete in the proportions of one part of cement to four parts of mixed aggregate. The same quantity of a well-graded concrete sand from Elgin, Illinois, was used in all tests. Sufficient water was used to produce a relative consistency of 1.10, equivalent to a slump of about 3 to 4 in. The same water-ratio, after correcting for absorption, was used for all aggregates. Concrete specimens were made for the determination of compressive strength and resistance to wear as measured by the Talbot-Jones Rattler Test. The compression tests were made on 6 x 12-in. concrete cylinders and the wear tests on 8 x 8 x 5-in. concrete blocks. The yield, expressed as number of volumes of concrete for 1 volume of mixed aggregate, was also measured. The specimens were cured under normal temperatures, 14 days in the moist room and the remainder in the air of the laboratory. Each value is the average of 5 tests made on different days. The tests were made at the age of 3 months.

For the aggregates the unit weight, specific gravity, absorption, resistance to wear by four modifications of the Deval Abrasion Test, and resistance to crushing were determined. The unit weights,[1] specific gravities,[2] and absorptions[3] were determined in accordance with methods described in the *Proceedings* of the American Society for Testing Materials.

The Deval Abrasion Tests and the crushing test were made as follows:

(1) In the A. S. T. M. *"Standard"* method, a 5,000-gram sample of 50 pieces, about 1½ to 2 in. in diameter, was placed in a cylinder of the Standard Deval machine, which was run for 10,000 revolutions. No abrasive charge was used.

(2) In the *"Lewis"* method, a sample consisting of 2,000 grams of ⅜ to ¾ in. material and 2,000 grams of ¾ to 1½-in. material was placed in the Deval machine with an abrasive charge of ten 1⅞-in. cast-iron balls and run for 2,000 revolutions.

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[1] See A. S. T. M. Standards, 1924, Page 759.
[2] See A. S. T. M. Standards, 1924, Page 928.
[3] See *Proceedings*, Am. Soc. for Testing Mat., Part I, 1920, Page 301.

(3) In the "*Rea*" method, the sample consisted of 2,500 grams of ⅜ to ¾-in. material and 2,500 grams of ¾ to 1½-in. material. The abrasive charge was six 1⅞-in. cast-iron balls and the number of revolutions was 10,000.

(4) In the "*Mattimore*" method, the procedure was the same as in

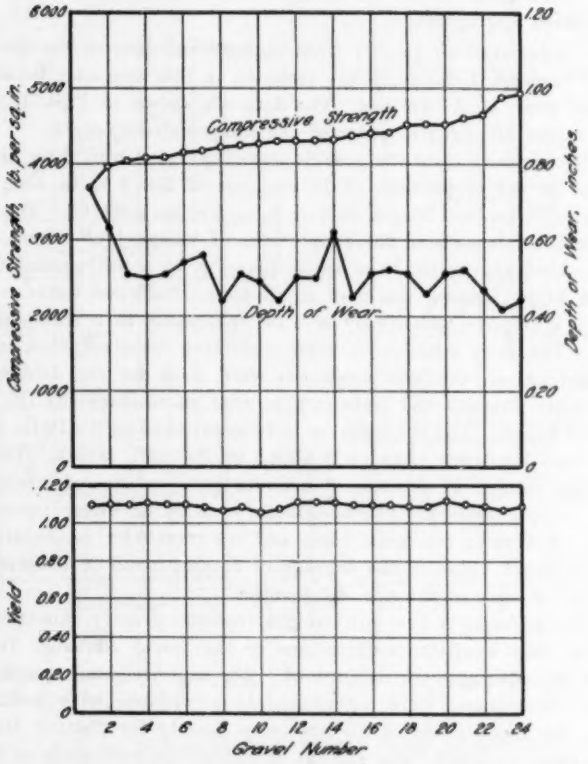

FIG. 1.—SUMMARY OF CONCRETE TESTS OF GRAVEL.

Data from Structural Materials Research Laboratory, Series 135, "Tests of Concrete made from Miscellaneous Coarse Aggregates."

the "Standard" method, except that the cylinder used was slotted to permit the escape of dust.

The values reported are the percentage of material passing a 16-mesh sieve after the test.

The Crushing Tests were made on 3,000-gram samples of coarse aggregates, graded as used in the concrete. The sample was placed in a cylindrical container, having a cross-sectional area of 30 sq. in. and a height

equal to the diameter. A load of 3,000 lb. per sq. in. was then applied in a testing machine and released immediately. The difference in fineness modulus as determined by sieve analyses before and after the test was taken as a measure of the crushing resistance of the aggregate.

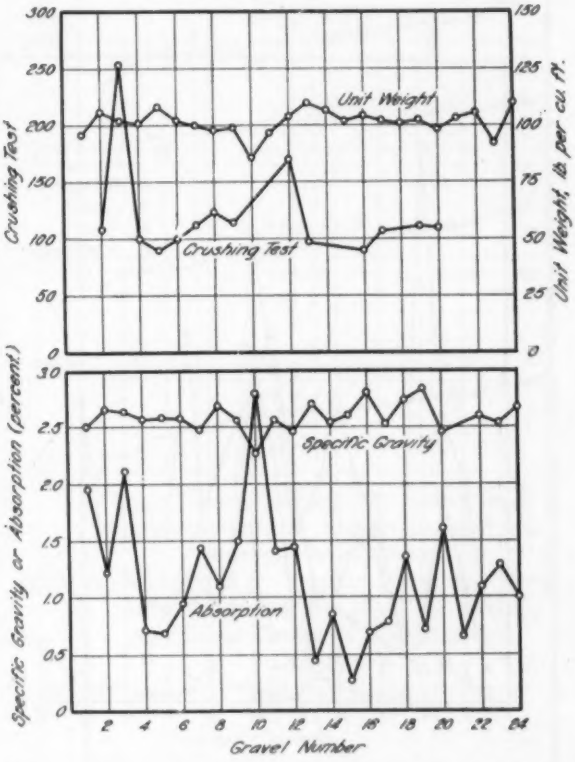

FIG. 2.—SUMMARY OF MISCELLANEOUS PHYSICAL TESTS OF GRAVEL.

Data from Structural Materials Research Laboratory, Series 135, "Tests of Concrete made from Miscellaneous Coarse Aggregates."

These tests, in common with those shown in Appendix 1, show no definite relation between compressive strength and the other factors measured.

NORMAL VARIATIONS IN CONCRETE TESTS.

In both Appendices 1 and 2 a considerable range in compressive strength is shown; in Appendix 1 from about 3,400 to 5,300 lb. per sq. in.

and in Appendix 2 from about 3,300 to 4,600 lb. per sq. in. A greater variation is shown in Appendix 1 than in Appendix 2, probably due to the fact that for Messrs. Jackson and Pauls' tests the grading of the different aggregates were not identical. It should be pointed out that a considerable proportion of these variations may be charged to normal variations in tests, without reference to the quality of the gravel. This is well illustrated

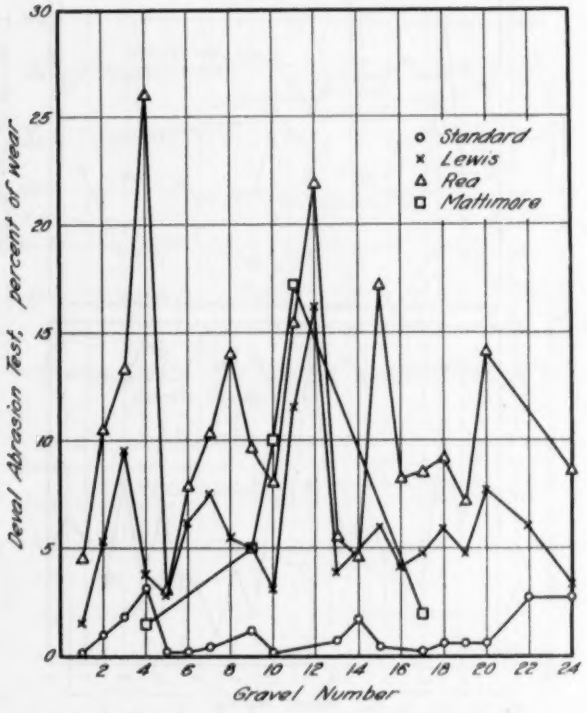

FIG. 3.—SUMMARY OF DEVAL ABRASION TESTS OF GRAVEL.

Data from Structural Materials Research Laboratory, Series 135, "Tests of Concrete made from Miscellaneous Coarse Aggregates."

by Fig. 4 (data furnished through the courtesy of Prof. D. A. Abrams), which shows the variation in compressive strength for values from twenty groups of five tests each. All of these were made from the same aggregates and under identical conditions, so far as they could be maintained. It will be noted that for concrete having strengths similar to those in Appendices 1 and 2, the variations shown by these tests are approximately 25 per cent of those shown by the tests in Appendix 1, and 30 per cent of those in Appendix 2.

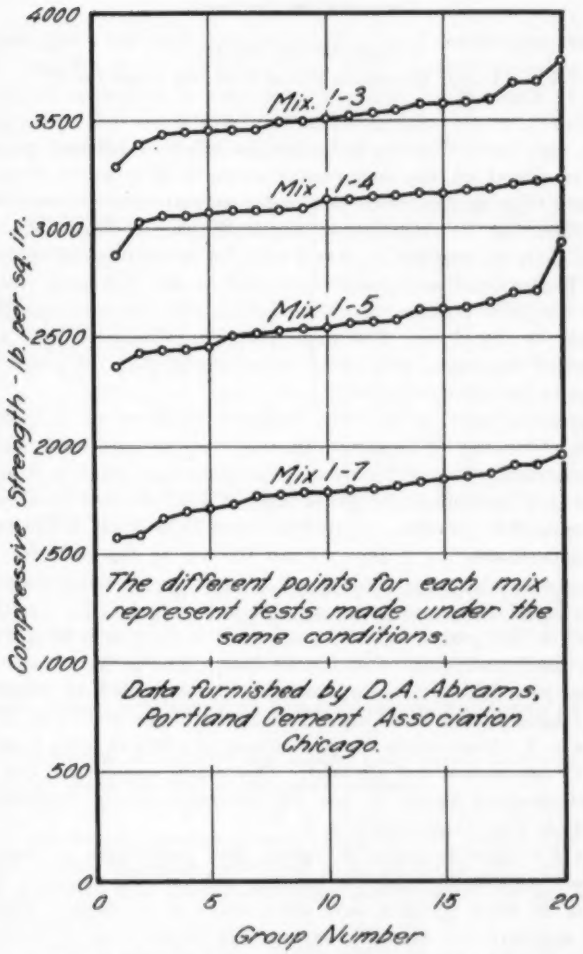

FIG. 4.—UNIFORMITY OF COMPRESSION TESTS OF 6 X 12 IN. CONCRETE CYLINDERS.

Data from Structural Materials Research Laboratory, Series 90, "Effect of Gage Length on Modulus of Elasticity of Concrete."

APPENDIX 3

Summary of Tests on Effect of Shale in Gravel on Compressive Strength of Concrete.

Tests carried out by
F. C. Lang, *Minnesota State Highway Department.*

TESTS.

Tests were carried out to determine the effect of different percentages of shale in gravel on the compressive strength of concrete cured under freezing and thawing conditions and under normal temperatures, with the view of obtaining information on which to base specifications for the amount of shale permissible in gravel used for concrete construction.

Well graded sand and gravel were used as the aggregate; the gravel was from the John Wunder Co., Minneapolis, and the sand was from the J. L. Shiely Co., St. Paul. The shale, from the Minnesota Pipe and Tile Co., Appleton, Minnesota, was added to the gravel with the proportions of different sizes the same as in the gravel.

Compression tests of 6 x 12-in. concrete cylinders of a 1:2:4 mix were made at the age of 28 days. Three specimens were made and tested for each condition. Four different series of tests were made as follows:

Series A.—Tests made for percentages of shale from 0 to 40 per cent, in increments of 4 per cent. Cylinders cured in a moist condition under normal temperatures for 7 days. From the 8th to the 18th day (eleven days) they were alternately stored out of doors in freezing temperatures for 24 hours and indoors in water at 25 deg. C. (77 deg. F) for 24 hours. At the end of this period they were stored in a room at a temperature of 25 deg. C. until tested at the age of 28 days. The outdoor temperatures, during the period that these specimens were subjected to freezing and thawing, ranged from 2 to 32 deg. F, with an average of 19 deg. F.

Series A-A.—Tests made for percentages of shale ranging from 0 to 8 per cent, in increments of 2 per cent. This series was carried out for the purpose of checking Series A, and all materials, curing conditions, etc., were identical with those in Series A.

Series B.—Same as Series A, except that percentages of shale up to 100 per cent were tested, and the freezing and thawing exposure was carried on for 26 days (from 2 days until tested at 28 days). During the period of exposure the outdoor temperatures varied from 1 to 44 deg. F and the average temperature was 28 deg. F.

Series C.—Same as A, A-A, and B, except that specimens were cured in a moist condition under normal temperatures from the time they were made until they were tested at 28 days.

Test Data.

Figs. 1 and 2, summarize the results of these tests. In Fig. 1 the compressive strengths in pounds per square inch are plotted. In Fig. 2

the compressive strengths are shown as strength-ratios, calculated as percents of the strength of concrete with no shale.

CONCLUSIONS.

Following are the conclusions which Mr. Lang draws from these tests:

"(1) Results of the foregoing tests show that the presence of shale even in small quantities is very detrimental to the strength of concrete, especially when the concrete is used in structures where it will be subjected to an alternate freezing and thawing action.

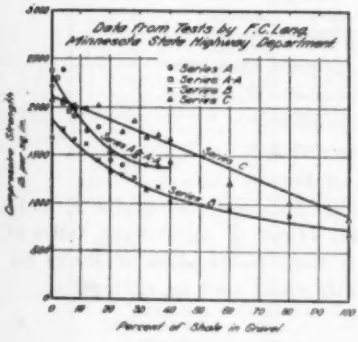
Data from Tests by F.C.Lang, Minnesota State Highway Department
Series A
Series A-A
Series B
Series C
Compressive Strength at 28 days, lbs. per sq. in.
Percent of Shale in Gravel

FIG 1

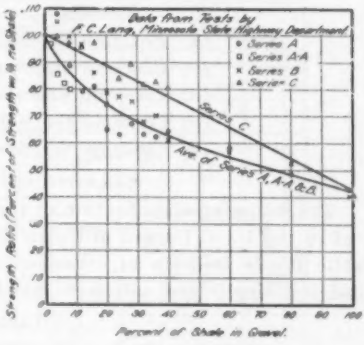
Data from Tests by F.C.Lang, Minnesota State Highway Department
Series A
Series A-A
Series B
Series C
Strength Ratio (Percent of Strength with 0% Shale)
Percent of Shale in Gravel

FIG. 2

FIG. 1.—EFFECT OF SHALE ON COMPRESSIVE STRENGTH OF CONCRETE.
Data from tests by F. C. Lang, Minnesota State Highway Department. Values are compressive strength in pounds per sq. in. at 28 days.

FIG. 2.—EFFECT OF SHALE ON COMPRESSIVE STRENGTH OF CONCRETE.
Same data as in Fig. 1, except that compressive strengths are expressed as "strength-ratios," with values for aggregate containing no shale as 100%.

"(2) Strength of concrete containing shale varies inversely with the percentage of shale the coarse aggregate contains.

"(3) The effect of weathering on shale is shown in the decreased strengths where test specimens were subjected to freezing and thawing. When the coarse aggregate contained more than 60 per cent shale the strength was very low and the freezing and thawing did not appear to further weaken it.

"(4) Concrete in which the coarse aggregate was 100 per cent shale showed a compressive strength in 28 days equal to about 40 per cent of the strength of concrete which contained no shale. It is probable that eventually such concrete would entirely disintegrate.

"(5) In these tests care was taken to see that there was no separation of the shale due to its lighter weight. This care could not be taken in ordinary construction, especially pavements."

Characteristics of Crushed Stone Coarse Aggregate.

Report of Sub-Committee on Crushed Stone of Committee E-5 of A. C. I.

Scope of the Report.

The intent of this report is to present information on the characteristics of crushed stone coarse aggregate and, so far as possible, the effects of those characteristics on the properties of concrete.

The subject matter is discussed under the following headings:

1. Classification of rocks.
2. General discussion of rock characteristics.
3. Physical tests of rocks.
4. Properties of crushed stone for concrete for various uses.
   - (a) Unexposed concrete subjected to static and impact stresses.
   - (b) Concrete exposed to the weather.
   - (c) Concrete highways—surface abrasion.
   - (d) Concrete exposed to chemical action.
   - (e) Concrete for water resistance.
   - (f) Concrete for fire resistance.

Classification of Rocks.

The following classification of rocks has been proposed by Dr. E. C. E. Lord of the U. S. Bureau of Public Roads (Dept. of Agriculture, Office of Public Roads Bulletin 31, "Examination and Classification of Rocks for Road Building") and will serve to classify rocks used as aggregates.

Table I.

General Classification of Rocks.

Class	Type	Family
I. Igneous	1. Intrusive (plutonic)	a. granite b. syenite c. diorite d. gabbro e. peridotite
	2. Extrusive (volcanic)	a. rhyolite b. trachyte c. andesite d. basalt and diabase
II. Sedimentary	1. Calcareous	a. limestone b. dolomite
	2. Siliceous	a. shale b. sandstone c. chert (flint)
III. Metamorphic	1. Foliated	a. gneiss b. schist c. amphibolite
	2. Nonfoliated	a. slate b. quartzite c. eclogite d. marble

The *igneous* rocks are supposed to have been formed by solidification from a molten state either at the earth's surface or at some distance below. Those solidified at the surface, the extrusive, igneous rocks, have cooled quickly, are rather fine grained, glossy and vesicular or have a porphyritic structure having large crystals in a fine grained ground mass. The intrusive or deep-seated igneous rocks have been solidified slowly

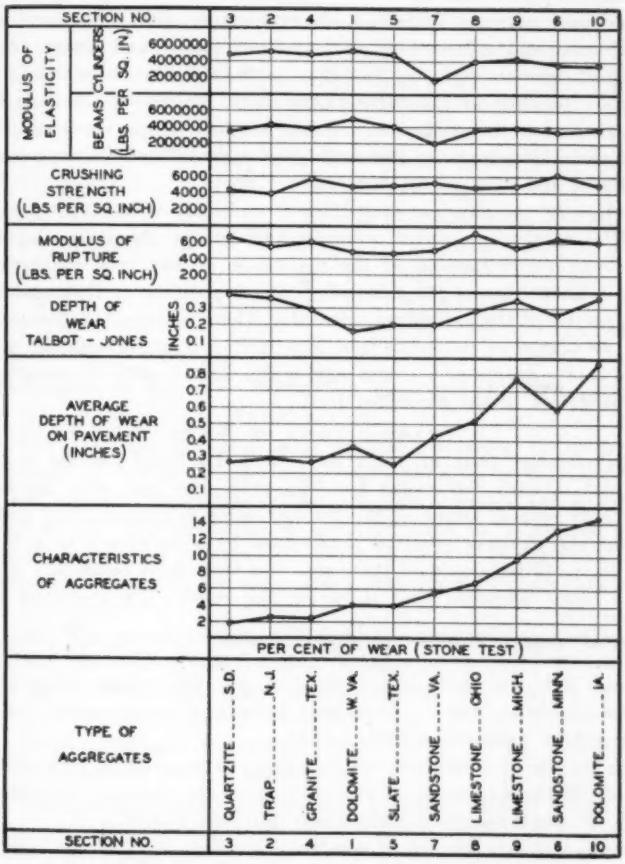

FIG. 1.—RESULTS OF PAVEMENT WEAR TESTS.

and under great pressure. They are generally coarsely crystalline with their constituent minerals well defined as contrasted with the fine grained extrusive rocks. The term "trap" from the Swedish word "trappa" meaning "stair" is commonly applied to the dark-colored igneous rocks such as gabbro peridotite, diabase, basalt and others. This term is also applied commercially to certain hard, dense rocks not of igneous origin.

The *sedimentary* rocks are made up through the consolidation either of the products of former rock disintegration as in the case of sandstone, shale, conglomerate, etc., or from the accumulation of organic remains of a calcareous nature forming limestone or dolomite. Sometimes these materials have been mechanically deposited, at others chemically precipitated as in the case of flint, chert, travertine limestones, etc.

Metamorphic rocks have been produced by the prolonged effect of chemical and physical agencies such as pressure, heat, moisture, etc., on both igneous and sedimentary rocks. Thus gneiss and schist are metamorphosed, igneous rocks while quartzite, marble and slate have resulted from an alteration of the sedimentary rocks. Some of the metamorphic rocks, such as slate and schist because of their shape after crushing, or due to their structure are not as highly regarded for use in concrete as the igneous and most of the sedimentary rocks.

Obviously the physical properties of rocks are influenced by the characteristics of the component minerals and by the structure of the rock. A thorough understanding of the resistance of any rock must therefore involve a knowledge of the mineralogical composition of that rock and of the properties of the individual minerals. There is some question, however, as to the degree of decomposition any rock will suffer after it is incorporated into concrete as a coarse aggregate, for evidently the mortar offers some protection to the embedded rock.

PHYSICAL TESTS OF ROCKS.

The physical tests of rocks have been developed for the most part in connection with their road building properties but none the less the tests are in general aimed at the determination of fundamental properties and thus the physical qualities determined by tests give a very fair idea of the degree of resistance of the rock. The tests most commonly employed are the Deval Abrasion Test in which the results are expressed as percentage of wear or french coefficient of wear ($= \frac{40}{\text{per cent wear}}$), the toughness test, the Dorry Hardness Test, the test for crushing strength and the absorption test. These tests have been described in the U. S. Department of Agriculture Bulletin 347.

The results of tests made on thousands of samples of rock show that rocks of all classes range considerably in their physical characteristics. (See U. S. Department of Agriculture Bulletin 370.)

Average results are not of any great value in view of the wide variation from the average to be expected from any particular sample of rock contemplated for a given use, and it thus becomes important to discuss the properties of rock for use as an aggregate from the standpoint of the service it is expected to render. Before leaving the subject of test results, however, the resistance of a rock to freezing and thawing or to some form of accelerated soundness test should be mentioned, as such tests throw considerable light on the suitability of a rock for use as a concrete aggre-

gate, though much remains to be done in formulating proper methods for making these tests and in establishing safe test limits.

CLASSES OF SERVICE TO BE RENDERED BY CONCRETE AND THE INFLUENCE OF THE COARSE AGGREGATE.

It is obvious that before any consideration can be given to the properties necessary for coarse aggregate it will be necessary to study the various kinds of service which concrete is called upon to render. This subject has already been treated (American Concrete Institute *Proceedings*, 1926, "What are the Most Significant Tests for Concrete?" by A. T. Goldbeck). It was pointed out that different structures have to withstand different combinations of stresses and that depending upon conditions of service, concrete is called upon to be resistant along the following lines:

1. Compression
2. Tension
3. Cross-bending
4. Shear
5. Impact
6. Surface abrasion
7. Chemical action
8. Freezing
9. Stresses from alternate wetting and drying
10. Absorption
11. Permeability
12. Heat resistance

THE PROPERTIES OF CRUSHED STONE FOR CONCRETE TO WITHSTAND THE ABOVE ACTIONS.

To arrive at the characteristics of stone suitable for concrete it seems entirely proper to consider these characteristics in the light of the service the concrete must render and the above twelve listed forms of resistance include practically all of the conditions to be met in service.

For the purpose of studying the effect of the coarse aggregate on the resistance of the concrete the above individual forms of resistance may be grouped as follows:

(a) Unexposed concrete subjected to static or impact stress
(b) Concrete exposed to the weather
(c) Concrete highways—surface abrasion
(d) Concrete exposed to chemical action
(e) Concrete for water resistance
(f) Concrete for fire resistance

(a) *Unexposed Concrete Subjected to Static or Impact Stress*

1. Compression
2. Tension
3. Cross-bending
4. Shear

It is very seldom the case that the crushing strength of stone proposed for use as aggregate is less than that of the strongest concrete made. It would seem logical that so long as the stone has a crushing strength at least equal to that desired of the concrete, the strength of the stone will not be a factor in influencing the strength of the concrete. Some illuminat-

ing results on the influence of the coarse aggregate on various properties of concrete are reported in an article entitled, "Wear of Concrete Pavements Tested," *Public Roads*, May, 1924, by F. H. Jackson and J. T. Pauls. Fig. 1 is plotted from results contained in this article. It will be noted that although the stone used in the 1: 1½: 3 mixture of concrete varied in per cent of wear (Deval Abrasion Test) from less than 2.0 to over 14.0 the crushing strength of all of the stone concrete was at least equal to 4000 lb. per sq. in. and there was no greater crushing strength shown by concrete containing the hard, tough rocks than by that made with the softest dolomite which had the following characteristics:

Per Cent Wear	Hardness	Toughness	Wt. per Cu. Ft. (solid)	Absorption	Grading
14.5	0.0	4	132	8.96	¼-1¼ in.

The crushing strength test was not made on this sample of stone but the above physical values show this rock to be lacking in both hardness and toughness and its crushing strength must also be low, yet the crushing strength of the concrete was apparently unaffected.

In a series of toughness tests made at the Bureau of Public Roads and reported by F. H. Jackson in the July, 1917, issue of the *Journal of Agricultural Research*, it was necessary to use a 2-in. cylinder of mortar 2 inches in height rather than the customary 1-in. x 1-in. cylinder as used in the toughness test for rock because all mortars were less tough than the weakest rocks ordinarily used commercially.

Prof. F. E. Giesecke, University of Texas, has published data (*Engineering News-Record*, June 29, 1922) showing that with crushed stone ranging in crushing strength from 4400 to 33,600 lb. per sq. in. the crushing strength of concretes containing 6, 8, 10, 12 and 14 sacks of cement per cu. yd. of concrete were practically identical. This was so even when the strength of the concrete greatly exceeded that of the aggregate.

It would seem that if the rock used as the coarse aggregate is as strong as the mortar portion of the mix the rock will certainly be amply strong. The following test limits are tentatively suggested as satisfying the above requirements:

Crushing Strength	Per Cent of Wear	Toughness
6000	8 (Fr. Coeff. 5)	5

Note: U. S. Bureau of Public Roads tests have shown the following results on soft rocks:

Sample Number	State	County	Material	Per Cent Wear	Toughness	Crushing Strength	Absorption, lbs. per cu. ft.
4197	Indiana	Lawrence	Argillaceous limestone	8.1	4	12,250	5.77
5027	Indiana	Lawrence	Limestone	6.4	4	6,900	2.97
5029	Indiana	Lawrence	Limestone	7.6	5	6,450	2.42

(See References.)

CROSS-BREAKING STRENGTH.

With regard to the influence of characteristics of crushed stone coarse aggregate on the cross-breaking strength of the concrete, reference may again be made to Fig. 1. Note that the quartzite with a percentage of wear of less than 2.0 produced concrete with a modulus of rupture of over 600 lb. per sq. in. (677) while the softest rock, a dolomite with 14.5 per cent wear made concrete having a modulus of rupture of 599 lb. per sq. in. while the concrete having the highest value for modulus of rupture, 729 lb. per sq. in. had a percentage of wear of 6.3. It is quite evident from the results shown in Fig. 1 that modulus of rupture of concrete made of stone is not a function of the resistance of the stone to abrasion in the Deval Abrasion Test. The dolomite above referred to is far softer than is generally permitted in highway construction and yet it is stronger than the matrix and hence the strength of the concrete has not been affected by its low resistance. This has an important bearing on concrete road specifications, for it is apparent that considering the cross-breaking strength of the concrete alone, the coarse aggregate can be at least as soft as the dolomite above referred to. There are, of course, other factors to consider in connection with aggregates for concrete roads.

TENSION.

No results have been found to show the influence of the characteristics of crushed stone coarse aggregate on the resistance of concrete to direct tension. However, it has been shown that the modulus of rupture is unaffected and since modulus of rupture and direct tension are closely related it can be safely assumed that the tensile strength will be no more affected by the different characteristics of crushed stone coarse aggregate than the modulus of rupture.

(b) *Concrete Exposed to the Weather.*—If in addition to static stress, concrete is to be exposed to the weather then not only should the stone be structurally strong enough but in addition it should be resistant to the effects of the weather. In general, the most rapid weathering proceeds through the disrupting action of frost and of crystalline deposits within the pores and such actions are possible only where water is permitted to be entrained within the concrete mass. It has been shown that limestone, sandstones with a calcareous binder and even other rocks might be subject to decay due to the disrupting effect of calcium sulphate formed by the combination of sulphur dioxide with calcium carbonate in the presence of air and moisture. In the case of rocks other than limestone or calcareous sandstone the lime necessary for the formation of calcium sulphate is supplied from the cement. High porosity would accelerate such action. (See *Rock Products*, Oct. 16, 1926, p. 94.) This applies more particularly to exposed building stone rather than to aggregate well protected with mortar.

Rocks having a high percentage of absorption and which at the same time are structurally weak are apt to weather rapidly. It is also claimed that there are some rocks which because of the presence of seams are apt to take up water and cause trouble through expansion of the concrete surface. Some types of shale also break down very quickly and an instance is noted by J. C. Pearson and G. F. Loughlin of the deleterious effects of a soda lime (labadorite) of the altered plagioclase variety. ("An Interesting Case of a Dangerous Aggregate," by J. C. Pearson and G. F. Loughlin, American Concrete Institute *Proceedings*, 1923). An attempt is made to detect all unsound aggregates either by a freezing and thawing test or by an accelerated soundness test such as the sodium sulphate test.

Other soundness tests which have been proposed are:

1. The sodium chloride test.
2. The alkali test.

(See *Proceedings* of the Highway Research Board, 1924, p. 109, "Soundness Tests for Coarse Aggregate," by M. O. Withey.)

A number of tests for soundness of rocks made by the Minnesota Highway Department are also reported in the 1924 *Proceedings* of the Highway Research Board but are not conclusive in showing any relation between physical characteristics of rocks and soundness.

It is quite evident that much remains to be done in the development of a truly satisfactory test for the soundness of coarse aggregates. In the meantime if a rock does not withstand five immersions in the sodium sulphate test, the only really satisfactory test is the appearance of the rock as it is exposed on the ledge. If five immersions are withstood the rock in all probability will be found to be satisfactory withstanding the weather and many rocks are also giving entire satisfactory service which do not withstand this test. It is suggested as a suitable requirement for the soundness of stone that it will be accepted provided an examination of the ledge or of structures in service shows it to be satisfactory or if it withstands five immersions in the sodium sulphate test.

(c) *Concrete Highways—Surface Abrasion.*—The Arlington Tests of the Bureau of Public Roads more nearly simulate the abrading effect of traffic on concrete roads than any other tests thus far made and the conclusions from these tests as they apply to stone concrete are as follows:

"1. That the rate of wear of stone concrete is, in general, not affected by the coarse aggregate provided the coarse aggregate is equal or superior to the mortar matrix in resistance to wear.

"2. That excessive wear will result from the use of very soft stone as coarse aggregate even though used in conjunction with a mortar of satisfactory quality. From the results of these comparative tests, it would appear that stone with a percentage of wear over 7.0 should not be used in the wearing course of concrete roads.

"6. That small amounts of shale occurring in the coarse aggregate will cause both excessive and uneven wear.

"15. That the Talbot-Jones wear test is not, in general, an indication of the wear which takes place under traffic.

"16. That neither the crushing nor the transverse strength of concrete is a measure of its wear-resisting properties."

In drawing the above conclusions the authors considered only the effects of wear when tire chains were used on the wheels. There was no abrasive effect whatever on any of the sections under the action of the equivalent of 150,000 vehicles passing over the same wheel track. The indications are that the actual abrasive wear of rubber tired traffic is a negligible factor. However, the above conclusions are probably generally applicable with exceptions where local economic conditions make such exceptions necessary. For instance, in some localities there is no hard rock, tire chains are little used and the climate is not cold. Here the above limit of 7 per cent wear might be raised at least to 8 per cent.

D. A. Abrams, Structural Materials Research Laboratory, Lewis Institute, Bulletin No. 10, "Wear Tests of Concrete" states in conclusion No. 10:

"(10). The quality of the fine or coarse aggregate produced less effect on wear than is commonly supposed. The wearing resistance of concrete is determined largely by the *quality of concrete* rather than by the type of aggregate. Good concrete can be produced from aggregates which are generally considered inferior, if other factors are properly taken into account."

Obviously where there is very severe abrasive wear to be withstood such as in factory buildings, railroad platforms, particular sidewalks and floors, pavements carrying steel tires or wheels equipped with chains, a hard, tough aggregate is desirable.

(d) *Chemical Action.*—Where concrete is subjected to chemical action the stone should be at least as resistant as the mortar portion of the concrete. For resistance to alkali effects the sodium sulphate test is directly indicative of the quality of the stone and the stone should be equally resistant as the mortar in the sodium sulphate test.

So far as resistance to acids is concerned the same statement should be made—the stone should be as resistant as the mortar and this can be tested on an accelerated scale at least as satisfactorily as in the case of the soundness test. Among the chemicals having a deleterious effect on concrete are alkali and sea-water, animal and vegetable oils, tanning liquors, sulphite liquor, vinegar, sugar solution, molasses and various kinds of acids.

It is suggested as a requirement for stone coarse aggregate for concrete exposed to chemical action: "The crushed stone shall be equally as resistant as the mortar portion of the concrete when exposed to any particular chemical action."

(e) *Water Resistance.*—The coarse aggregate might affect the water resistance of concrete:

1. By its effect on permeability.
2. By its effect on the durability of the concrete.

1. *Permeability.*—A very comprehensive series of tests was reported by Prof. M. O. Withey in Bulletin No. 1245, Engineering Series, Vol. IX, No. 2, of the University of Wisconsin. It is shown by him that no matter what kind of coarse aggregate is used there is a fairly definite relation between permeability and crushing strength. Properly cured broken stone concrete of plastic consistency is shown to be impermeable when the crushing strength is 2500 lb. per sq. in. The grading of the aggregate to promote maximum density is also important and grading according to Fullers' curve is said to be beneficial. The characteristics of the concrete rather than the characteristics of the aggregate seem to be most important in producing impermeable concrete. However, soundness is an important characteristic and should be required of stone subjected to water action.

2. *Resistance of Aggregates to Water Action.*—If the concrete is impermeable, water action then becomes primarily a surface phenomenon, except in the case of very porous and absorbent concrete. Its effects are chemical and physical. So far as the dissolving action of water is concerned, this is negligible on stone which will withstand disintegration in the weather. The percentage of absorption of the mortar matrix is so far in excess of even the softest, *sound* limestone aggregate that there is little likelihood of the aggregate ever being more soluble than the mortar. Stone aggregate subjected to water action should be sound and have reasonably low absorption for good results in concrete subjected to water action. There seems to be little to fear from the effect of freezing action on the stone in concrete provided the stone is sound, for the mortar is invariably weaker and of much higher absorption than any stone that will withstand the weather. Suggested special requirement:

1. Shall be sound as demonstrated by appearance of ledge or by successful use in structures or by accelerated soundness test.

(f) *Concrete for Fire Resistance.*—Extensive work has been done on the fire resistance of concrete as influenced by the coarse aggregate. This work has been summarized by Committee E-4 on Fire Resistance of Concrete by the American Concrete Institute and that portion of the "Résumé" dealing with aggregates is as follows:

RESUME.

"The lessons that may be drawn from the studies so far made by the committee are that the fire resistance of concretes depends to a great extent on the kinds of aggregates used. Aggregates, such as the siliceous

gravels used in the tests reviewed, result in concretes which are likely to spall rather quickly when exposed to fires. Small percentages of chert or other highly siliceous aggregates mixed with aggregates which do not spall may still cause serious cracking and spalling. Sandstones and granites vary somewhat in affecting the fire resistance of concretes in which they are used as aggregates, but usually the results are slightly better than with the siliceous gravels. Both have the tendency to crack and spall. Hard-coal-cinder concrete does not show this tendency but transmits heat more readily, therefore, does not give longer protection to the steel and structural concrete. Concrete made from blast furnace slag gives results in fire tests about equal to those of trap rock concrete. Both are decidedly better than concretes having the highly siliceous aggregates. In nearly all tests limestone has been shown to be superior to all the other natural aggregates in its fire resisting qualities. There is little or no tendency for the limestone concrete to spall or crack and its insulating value is generally greater. In fires of long duration the limestone aggregate near the surface becomes calcined and in some cases necessitates more surface repair to the protective covering than where trap rock is used, but these cases are the exception rather than the rule. So far as tests have been made it has been found that for rocks of a given mineral composition those of coarsely crystalline structure are not as resistant to fire as those of fine structure. Broken bricks or burnt clay aggregates give favorable results in strength and fire resistive properties."

All of the foregoing suggestions as to special requirements in specifications for stone are to be regarded as tentative, for they are based on rather meagre information. As additional data are accumulated more definite recommendations will be made possible.

PROBLEMS TO BE SOLVED IN CONNECTION WITH SPECIFICATIONS FOR STONE AS A COARSE AGGREGATE IN CONCRETE.

1. What is the effect of dust-coated stone on the strength of concrete?
2. What is the allowable percentage of dust?
3. Define stone dust and standardize a method for its determination.
4. Standardize a truly significant soundness test for all aggregates.
5. What are desirable gradings for stone to produce?
    a. Greatest workability of concrete.
    b. Greatest yield.
    c. Greatest strength.
6. What is the effect of unsound particles and how large a percentage is permissible?
    a. In pavements.
    b. In exposed structures.
7. What is the effect of "flat" and "elongated" pieces and what percentage should be allowed?
8. What is a proper definition for "flat" and for "elongated"?

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A. T. GOLDBECK, *Chairman*.

H. S. MATTIMORE,

R. L. BERTIN,

H. E. BREED.

Requirements of Slag as an Aggregate for Concrete.

Report of Sub-Committee on Slag of Committee E-5 of the American Concrete Institute.

Acting on instructions received Aug. 4, 1926, from R. W. Crum, chairman, Committee E-5, the Sub-Committee on Slag began assembling information and data on air-cooled blast furnace slag for all kinds and classes of concrete.

Although sincere effort has accomplished a relatively large amount of work, the committee feels that it has not been allowed sufficient time to cover, in an authoritative and exhaustive manner, the fields outlined.

That such condition obtains is due largely to the fact that there is apparently a wide range of opinion in the testing profession as to the value of certain tests on mineral aggregates. The committee, therefore, frankly leans towards placing their approval on tests of the concrete in which slag is the aggregate as the measure of the value of the material.

This after close observation of a great deal of the exposed portion of hundreds of examples of the tabulated thirty millions of cubic yards of slag concrete and reinforced concrete, some of which now possess service history up to twenty-five (25) years. As an indication of the sincerity of the attack on the problem, it may be stated that a part of the exising literature has been "analyzed and digested" under the following classifications:

Abrasion	Elasticity	Sea Water
Abrasive Qualities	Expansion	Shape
Absorption	Fineness Modulus	Shear
Organic Matter	Fire	Size
Percussion	Flow Sheet	Solubility
Permeability	Fracture	Soundness
Bibliography	Geography	Specifications
Bond	Gradation	Specific Gravity
Bulking	Hardness	Stability
Cementation	History	Strength
Chemical	Homogeneity	Surface Area
Cleanliness	Impact	Temperature
Color	Melting Point	Tensile
Compression	Moisture	Texture
Compressibility	Accessibility	Thermo-Conductivity
Corrosion	Acoustics	Toughness
Corrosive Qualities	Age	Uniformity
Definition	Petrographic	Uses
Density	Possibilities	Voids
Desirability	Processing	Weathering
Durability	Production	Weight
Economy	Research	Workability
Efficiency of Tests	Rupture	Yield

Early in the work of the committee it was decided that a specification could not be developed or maintained without a most comprehensive survey of what had been written about the material for all uses to which it had been placed. Seemingly no detail of production, processing, chemical analysis, physical limitations, service history or laboratory data of any sort could be discarded as worthless or inconsequential. For this reason the work accomplished, and further outlined, involves the study of much more information than was indicated as necessary in the original instructions to the committee.

We would, therefore, ask for an extension of time in which to complete our work during which the present status of slag in Committee E-5's Tentative Specification remains unchanged.

H. J. LOVE, *Chairman.*
J. J. PAINE,
F. H. JACKSON,
FRED HUBBARD.

Concrete Products Plant Operation.

Reported by Committee P-6.

The feature of this year's work has been the completion of tests to determine the effect of weather conditions on outdoor curing of concrete masonry units. Block were made each month during the year, beginning November, 1925, and tested at 28 and 90 days after the usual initial curing followed by storage in the open. These tests were made through the co-operation of the Diamond Block Co., St. Paul, Minn., and the structural division of the civil engineering department, University of Minnesota, and the test program was conducted under the direction of Earle D. McKay, a member of this committee. The results of this investigation are given in Appendix 1 of this report.

The work of this committee for the coming year will consist largely of collecting information upon methods for increasing plant operation efficiency. The committee felt a distinct need for information of this nature. Accurate data as to labor hours required for each plant operation will be secured. A form (Appendix 2) was prepared for use of the manufacturers in submitting this information and in order that the data may be readily analyzed by the committee. A comparison of the statistics obtained from as many manufacturers as possible will be used in arriving at reasonable labor requirements for the different steps in the manufacturing process.

APPENDIX 1.

Effect of Weather Conditions on Curing of Concrete Block.

Purpose.—This investigation, which extended over a 12-month period, was made for the purpose of studying the effect of outdoor storage on the strength and absorption of concrete building block made at various seasons of the year. Although it is usual in ordinary practice to store block in the yard after a brief period of indoor curing, no particular study had heretofore been made to determine the effect of weather conditions during the yard storage on the strength of the block.

In October, 1925, the sub-committee on curing of Committee P-6 outlined a series of tests in order to secure data on this subject. The investigation was begun in November, 1925, and continued for one year.

Acknowledgment.—The test block were manufactured and stored at the plant of the Diamond Block Co., St. Paul, Minn. The hearty co-operation of everyone connected with the plant, particularly that of L. C. Christensen, manager, made the work a pleasure as well as a success.

The committee is indebted to the structural division of the civil engineering department of the University of Minnesota in whose laboratories the block were tested under the direction of Prof. M. B. Lagaard or Frank E. Nichol. The investigation was conducted under the personal supervision of Earle D. McKay, Universal Portland Cement Co., Minneapolis, assisted by R. Conrad Cooper of the same company.

Outline of Tests.—An outline of the tests covered by the investigation is given in Table I. A total of 240 8 x 8 x 16-in. block were tested.

MATERIALS AND METHODS.

Materials.

Cement.—Twenty-five sacks of Universal portland cement were delivered by a St. Paul dealer to the plant where it was immediately emptied into metal cans, which were sealed and placed in a dry, warm part of the factory. No cement other than this was used. Every 3 months a sample was tested, which, compared with the original mill test, showed practically no change in properties during the period of this investigation.

Aggregates were obtained from a St. Paul dealer, who delivered a sufficient quantity for the tests, and consisted of:

Washed sand	0 to No. 4
Limestone screenings	0 to No. 4
Pea gravel	No. 4 to ⅜ in.

TABLE I—OUTLINE OF TESTS ON CURING OF CONCRETE BLOCK

Compression tests of 8 × 8 × 16-in., 3-core concrete building block.
Mix 1:7 by volume of dry and rodded combined aggregates.
Cement: portland.
Aggregate: a mixture of 0 to No. 4 sand, 0 to No. 4 limestone screenings and No. 4½-in. pebbles; fineness modulus of mixed aggregate 4.15.
Consistency as wet as practicable; mixing water accurately measured.
Machine-mixed concrete (½-bag batches); concrete mixed 3 min. after all materials, including water, were in mixer.
Block tamped the same number of blows and made by same operator.
Four block selected for test for each condition from at least 3 batches.
Block cured as indicated; block, while outdoors, protected from sun by additional block.

Group	Month Made	Average Outdoor Temperature for Month Made, deg. F.	Initial Curing Condition	Age at Test	Number of Specimens	
					Made	Tested
1	June (1926)	71	(1) In moist room for 24 hr., then outdoors	28 days	135	80
	July (1926)[a]	69				
	August (1926)[b]	63	(2) In steam room at 100 to 125° F. for 24 hr., then outdoors	3 mo.		
	September (1926)	50				
2	October (1926)[c]	31	(3) In moist room at about 70° F. for 24 hr., then outdoors	28 days	108	64
	April (1926)[a]	58				
	May (1926)	65	(4) In unheated moist room for 24 hr., then outdoors	3 mo.		
3	November (1925)[a]	32	(5) In moist room at about 70° F. for 24 hr., then outdoors	28 days	162	96
	December (1925)[d]	25				
	January (1926)	15	(6) In steam room at 100 to 125° F. for 24 hr., then outdoors	3 mo.		
	February (1926)	22				
	March (1926)	34				
				Total	405	240

[a] Absorption tests made on block.
[b] (1) and (2) repeated during August; block stored in Laboratory at about 70° F. after initial curing.
[c] (3) and (4) repeated during October; block stored in Laboratory at about 70° F. after initial curing.
[d] (5) and (6) repeated during December; block stored in Laboratory at about 70° F. after initial curing.

TABLE II—SIEVE ANALYSIS AND MISCELLANEOUS TESTS OF AGGREGATE

Sieve Analysis

Number or Size of Sieve	Size of Square Opening, inches	Amount Coarser than Each Sieve, per cent by weight			
		Sand	Limestone Screenings	Pea Gravel	Combined Aggregate
100	0.0058	99.0	91.0	100.0	96.0
48	0.0117	93.2	80.0	100.0	93.0
28	0.0232	62.0	67.0	100.0	79.0
14	0.0469	32.4	48.4	100.0	60.0
8	0.093	16.0	26.4	99.0	45.0
4	0.185	3.8	1.6	68.0	31.0
⅜ in.	0.37	0.0	0.0	30.4	11.0
¾ in.	0.75	0.0	0.0	0.0	0.0
Fineness Modulus*		3.07	3.14	6.17	4.15

* Sum of the percentages in the sieve analysis divided by 100.

Miscellaneous Tests

	Sand	Limestone Screenings	Pea Gravel	Combined Aggregate
Unit weight, lb. per cu. ft., dry and rodded	112	107	104	122
Organic impurities, by colorimetric test	none	none	none	...
Silt, per cent by weight	0.2	3.2	none	...

These materials were stored separately and covered with tarpaulins to protect them from the weather.

The values for sieve analysis and miscellaneous tests of aggregate in Table II are the average of 12 determinations, a complete test being run on the aggregate each month prior to making the block.

Water for mixing was from the St. Paul city supply.

Miscellaneous Tests.

	Sand	Limestone Screenings	Pea Gravel	Combined Aggregate
Unit weight-lb. per cu. ft., dry and rodded	112	107	104	122
Organic impurities, by colorimetric test...	None	None	None	
Silt, per cent by weight.....	0.2	3.2	None	

Concrete.—The proportions of cement to mixed aggregate were 1: 7 by volume of dry and rodded materials. The aggregates were measured separately. The amount of each aggregate (based on dry and rodded volumes) per sack of cement was as follows:

2 cu. ft. of limestone screenings	weighing	107	lb. per cu. ft.	=	214 lb.
3 " " " sand	"	112	" " " "	=	336 "
3 " " " pea gravel	"	104	" " " "	=	312 "
			Total	=	862 lb.

A cubic foot of dry materials mixed in these proportions weighed 122 lb.; hence, $\frac{862}{122} = 7.07$ was the mix used—equivalent to 1 part cement, 2 parts screenings, 3 parts sand, 3 parts pea gravel by separate volumes.

The calculated fineness modulus of the combined aggregate was 4.26, but the average value for the 12 monthly determinations was 4.15 (Table II).

Each month, on the day on which the block were to be made, a moisture test was made on each aggregate in order to determine the amount of water in the aggregate and the amount of bulking resulting from the water in aggregate and from the method of measurement.

The following computations for November illustrate the method of obtaining the moisture content and bulking of aggregate:

Weight of sand damp and loose = 194 oz.
" " " dry = 185 "
Diff. = 9 oz.

$$\text{Moisture} = \frac{9}{185} = 4.86 \text{ per cent by weight of dry sand.}$$

Measurement of damp loose sand in container = 7.75 in.
" " dry rodded " " " = 6.13 "

Diff. = 1.62 in.

$$\text{Bulking} = \frac{6.13}{1.62} = 26.8 \text{ per cent by volume of dry, rodded sand.}$$

The moisture content and bulking of aggregate for November was:

Material	Moisture, per cent by weight	Bulking, per cent by volume
Sand	4.86	26.8
Screenings	3.10	24.0
Pea gravel	1.98	6.9

Table III gives a summary of proportions used each month, which were calculated as above indicated.

TABLE III—QUANTITIES OF MATERIALS USED PER BATCH EACH MONTH
Mix: 1:7 by volume of combined aggregates, dry and rodded.

Month	Cement		Sand		Screenings		Pea Gravel		Water						Water-Cement Ratio	Blocks per Sack of Cement
									In Aggregate		Added		Total			
	Wt. per lb.	Vol. per cu ft.	Wt. per lb.	Vol. per cu. ft.	Wt. per lb.	Vol. per cu. ft.	Wt. per lb.	Vol. per cu. ft.	Wt. per lb.	Vol. per gal.	Wt. per lb.	Vol. per gal.	Wt. per lb.	Vol. per gal.		
Nov.	47	½	168	1.90	107	1.24	156	1.60	14.6	1.75	23.5	2.81	38.1	4.56	1.22	18
Dec.	47	½	168	2.00	107	1.31	156	1.64	12.6	1.51	25.5	3.05	38.1	4.56	1.22	18
Jan.	47	½	168	2.00	107	1.31	156	1.64	12.6	1.51	25.5	3.05	38.1	4.56	1.22	18
Feb.	47	½	168	1.95	107	1.27	156	1.62	12.4	1.49	25.7	3.08	38.1	4.56	1.22	18
March	47	½	168	1.90	107	1.28	156	1.62	12.5	1.50	25.6	3.06	38.1	4.56	1.22	18
April	47	½	168	1.90	107	1.28	156	1.62	12.5	1.50	25.6	3.06	38.1	4.56	1.22	18
May	47	½	168	1.87	107	1.29	156	1.60	12.5	1.50	25.6	3.06	38.1	4.56	1.22	18
June	47	½	168	1.87	107	1.29	156	1.60	12.5	1.50	25.6	3.06	38.1	4.56	1.22	18
July	47	½	168	1.87	107	1.29	156	1.60	12.5	1.50	25.6	3.06	38.1	4.56	1.22	18
Aug.	47	½	168	1.85	107	1.31	156	1.63	9.2	1.10	28.9	3.46	38.1	4.56	1.22	18
Sept.	47	½	168	1.85	107	1.31	156	1.63	9.2	1.10	28.9	3.46	38.1	4.56	1.22	18
Oct.	47	½	168	1.80	107	1.25	156	1.57	9.0	1.08	29.1	3.50	38.1	4.56	1.22	18

The volume of damp and loose aggregate to be used was determined by multiplying the volume of dry and rodded aggregates desired by a factor representing the bulking effect. For example, in a ½-sack batch, 1.5 cu. ft. of dry rodded sand was used. For November, the sand was found to bulk 26.8 per cent, therefore, the volume of loose, damp sand used was 1.5 x 1.268 = 1.90 cu. ft.; similarly for the other aggregates.

The amount of water carried by the aggregate was calculated by multiplying the dry weight of aggregate used by the per cent of moisture. For example, in November the following amounts of water were carried by the aggregates:

Sand ..... 1.5 x 112 lb. x .0486 = 8.2 lb.
Screenings ..... 1.0 x 107 " x .0310 = 3.3 "
Gravel ..... 1.5 x 104 " x .0198 = 3.1 "

Total = 14.6 lb.

Equipment.—The materials were mixed in a "Blystone" mixer, elevated to the hopper of the machine by a drag elevator and belt conveyor. The block were made on a Papke vertical stripper machine, equipped with automatic tampers. Power was supplied by electric motors.

Measuring of Materials.—The aggregates were carefully measured in a 1-cu. ft. graduated box. The added water was measured in a graduated gallon container.

Mixing.—The aggregate and cement were first placed in the mixer and mixed for about 1 min.; then the water was added and materials were mixed for 3 min. During the winter months, the aggregate and water were heated to 70 deg. F. before being placed in the mixer.

Tamping.—The regular operator ran the block machine and the same number of tamps (9) were given each block.

Marking.—The block were marked consecutively as made using letters and numbers to denote month, batch, etc. Thus, "N B 6" means:

N—Made in November.

B—The second batch made

6—The sixth block made from a particular batch.

This system of marking was followed throughout.

Curing of Specimens.—Two methods of curing the block during the first 24 hr. were used as follows:

1. *Steam Room:* Immediately after making, the odd-numbered block were placed in a steam room which was kept as near 125 deg. F. as possible.
2. *Moist Room:* All even-numbered block were placed in a moist room which was kept at approximately 70 deg. F. for 24 hr.

The test block, except those sent to the laboratory for storage (Table V), were removed from the curing room after 24 hr. and stored in the yard. They were piled six high with cores vertical and covered at the top by other block laid with cores horizontal.

At the end of the curing period (28 or 90 days) the block were transported by truck to the experimental building, University of Minnesota, for test.

In order to obtain information on the effect of indoor storage on compressive strength, 48 block were made during the months of December (1925), July (1926), and October (1926), and after 24 hr. initial curing in moist room or steam room they were stored in the experimental laboratory at the University of Minnesota until tested at 28 and 90 days (Table V).

TESTING OF SPECIMENS.

Age.—Half of the block were tested when 28 days old and the other half at 3 months.

Treatment.—Due to lack of facilities for handling such a mass of specimens, it was impracticable to dry the block to constant weight. Consequently, the treatment adopted prior to testing was to dry the block for 3 days in the experimental engineering laboratory. The block were capped

with plaster of paris in accordance with specifications of the American Concrete Institute.

Method of Testing.—The block were allowed to stand approximately 1 hr. from time of capping until they were placed in the compression machine and tested. A 200,000-lb. Olsen testing machine was used. The total load on the block at failure can be reduced to load per square inch of gross or minimum area by dividing by 122 or 71, respectively.

Twenty-four block molded during January, July, and October, were tested for absorption in accordance with the specifications of the American Concrete Institute.

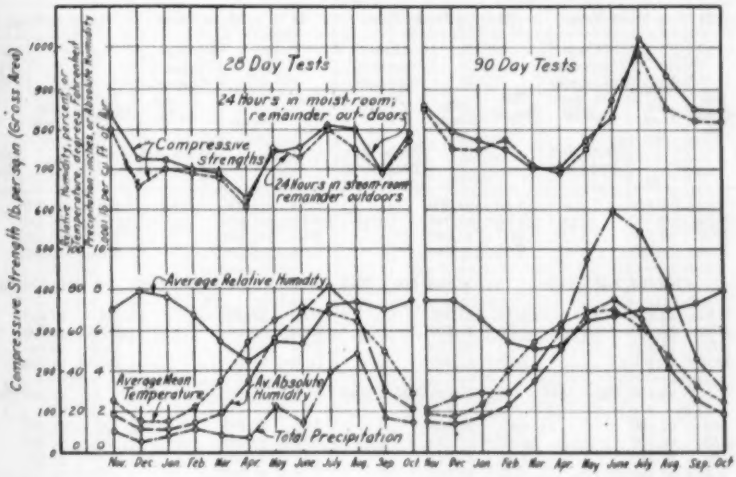

28 Day Tests
90 Day Tests
24 hours in moist room; remainder out-doors
Compressive strengths
24 hours in steam room remainder outdoors
Average Relative Humidity
Average Mean Temperature
An Absolute Humidity
Total Precipitation
Compressive Strength lb. per sq in (Gross Area)
Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sep. Oct Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sep. Oct

FIG. 1.—RELATION OF COMPRESSIVE STRENGTH TO METEOROLOGICAL CONDITIONS.

DATA AND DISCUSSION OF TESTS.

The data of the tests are given in Tables II to VI and in Fig. 1.

The 28- and 90-day compressive strengths of block cured in moist room or in steam room for the first 24 hr. and remainder outdoors are given in Table IV. Table V gives 28- and 90-day compressive strengths of block cured indoors in air of laboratory after 24 hr. in moist room or steam room at plant.

Table VI gives a summary of the meteorological conditions under which the block were cured. For a period of 1 month temperature readings were taken at the plant, adjacent to the test block, and compared with those of the St. Paul and Minneapolis stations of the U. S. Weather Bureau for the same period. The difference in the readings reported by the Weather Bureau and those taken at the plant was so small that readings at the plant were discontinued.

TABLE IV—TESTS OF CONCRETE BUILDING BLOCK

Compression tests of 8 × 8 × 16-in., 3-core concrete building block.
Gross area of block, 122.1 sq. in.; minimum area 71 sq. in.
Mix 1:7 by volume (based on dry and rodded aggregates).
Cement: portland.
Aggregate: mixture of 0 to No. 4 sand, 0 to No. 4 limestone screenings and No. 4¾-in. gravel; fineness modulus of mixed aggregate 4.25.
Consistency: as wet as practicable.
Block made on Papke vertical stripper machine equipped with automatic tampers.
Block cured for 24 hr., in moist room at about 70° F. or in steam room at about 125° F., then outdoors; block air-dried in laboratory 3 days before test. (See Fig. 1.)

Date Made	28-Day Tests								90-Day Tests							
	Initial Curing								Initial Curing							
	24 hr. in Moist Room				24 hr. in Steam Room				24 hr. in Moist Room				24 hr. in Steam Room			
	Reference No.	Breaking Load, lb.	Compressive Strength, lb. per sq. in.		Reference No.	Breaking Load, lb.	Compressive Strength, lb. per sq. in.		Reference No.	Breaking Load, lb.	Compressive Strength, lb. per sq. in.		Reference No.	Breaking Load, lb.	Compressive Strength, lb. per sq. in.	
			Gross Area	Net Area			Gross Area	Net Area			Gross Area	Net Area			Gross Area	Net Area
Nov. 18, 1925	NA2	100,600			NA3	104,900			NA4	111,400			NA7	99,070		
	NB4	106,740			NB5	91,540			NB6	107,420			NB3	108,200		
	NC6	98,640			NC7	91,240			NC4	104,400			NB7	98,860		
	NA8	106,780			NA5	101,760			NC8	98,680			NC3	105,680		
Aver.		103,190	845	1455		97,360	800	1370		105,480	865	1485		102,950	845	1450
Dec. 19, 1925	DA4	91,820			DA5	82,200			DA2	88,980			DA7	87,620		
	DB4	87,680			DB3	70,500			DB8	88,480			DB5	98,000		
	DB6	88,200			DB7	86,700			DC4	106,880			DC3	98,740		
	DC2	86,100			DC5	83,500			DC6	99,640			DC7	84,760		
Aver.		88,450	725	1245		80,730	660	1140		96,000	785	1350		92,280	755	1300
Jan. 18, 1926	JA4	82,640			JA5	86,460			JA6	87,400			JA3	81,420		
	JA8	90,600			JA7	91,280			JB4	89,200			JB7	90,640		
	JB2	87,740			JB3	83,280			JB8	100,220			JC3	95,330		
	JC6	92,260			JC5	80,540			JC4	94,460			JC7	96,870		
Aver.		88,310	725	1245		85,390	700	1200		92,820	760	1310		91,070	745	1285
Feb. 20, 1926	FA8	85,400			FA7	88,200			FA4	88,240			FA5	90,520		
	FB2	88,900			FB5	75,900			FB6	91,120			FB3	97,060		
	FB4	79,200			FC5	82,060			FC4	97,120			FB9	97,220		
	FC2	87,500			FC9	85,760			FC8	89,150			FC7	87,600		
Aver.		85,250	700	1200		82,980	680	1170		91,410	750	1290		93,100	765	1310
Mar. 21, 1926	MA8	92,320			MA3	86,580			MA6	100,720			MA9	96,900		
	MB6	80,220			MA7	74,940			MB4	73,420			MB5	85,340		
	MC2	79,820			MB3	86,280			MB8	79,580			MB7	86,300		
	MC4	79,060			MC7	77,600			MC6	87,480			MC5	81,420		
Aver.		82,860	680	1170		81,350	665	1145		85,300	700	1200		87,490	715	1230
Apr. 21, 1926	AA6	78,550			AA3	81,300			AA2	98,960			AA5	89,250		
	AB2	71,110			AB5	74,300			AA8	84,900			AB9	81,800		
	AB8	74,600			AB7	67,260			AB4	74,780			AC3	85,190		
	AC4	85,600			AC7	75,300			AC6	88,700			AC5	77,640		
Aver.		77,460	635	1090		74,540	610	1050		86,840	710	1225		83,470	685	1175

TABLE IV—TESTS OF CONCRETE BUILDING BLOCK—(Continued)

Date Made	28-Day Tests								90-Day Tests							
	Initial Curing								Initial Curing							
	24 hr. in Moist Room				24 hr. in Steam Room				24 hr. in Moist Room				24 hr. in Steam Room			
	Reference No.	Breaking Load, lb.	Compressive Strength, lb. per sq. in.		Reference No.	Breaking Load, lb.	Compressive Strength, lb. per sq. in.		Reference No.	Breaking Load, lb.	Compressive Strength, lb. per sq. in.		Reference No.	Breaking Load, lb.	Compressive Strength, lb. per sq. in.	
			Gross Area	Net Area			Gross Area	Net Area			Gross Area	Net Area			Gross Area	Net Area
May	MA6	87,980			MA3	78,020			MA4	103,980			MA5	93,800		
21,	MB2	88,800			MB7	96,300			MB6	86,580			MB3	88,300		
1926	MC2	90,180			MB9	93,200			MC4	96,060			MC7	90,700		
	MC8	92,080			MC5	100,760			MC6	87,420			MC9	91,680		
Aver.		89,760	735	1265		92,070	755	1295		93,510	765	1320		91,120	745	1285
June	JA6	83,200			JA5	83,600			JA4	97,940			JA3	104,500		
23,	JB4	106,420			JB3	97,400			JA8	109,700			JA5	112,780		
1926	JB6	86,000			JC3	94,060			JB8	100,160			JB5	104,100		
	JC8	97,000			JC5	85,280			JC6	101,280			JC7	104,740		
Aver.		93,160	765	1310		90,090	740	1270		102,270	840	1440		106,530	875	1500
July	JA2	100,540			JA3	90,150			JA4	117,620			JA7	126,600		
25,	JA8	96,160			JA5	116,700			JB6	114,840			JB3	126,460		
1926	JB4	112,300			JB5	99,700			JC2	130,060			JC5	121,520		
	JC6	91,440			JC7	89,800			JC4	133,760			JC3	108,480		
Aver.		100,110	820	1410		99,090	810	1395		124,070	1015	1750		120,770	990	1700
Aug.	AA2	107,100			AA3	92,760			AA6	115,700			AA7	121,380		
26,	AA8	93,500			AA5	96,400			AA2	98,980			AB7	98,040		
1926	AB4	98,780			AB5	84,340			AB6	116,600			AC3	101,020		
	AC6	90,060			AC7	98,300			AC4	125,500			AC5	91,820		
Aver.		97,360	800	1370		92,950	760	1310		114,200	935	1610		103,070	845	1450
Sept.	SA4	84,420			SA5	78,920			SA2	115,470			SA3	96,420		
28,	SB8	92,100			SB7	81,880			SB4	96,780			SA7	101,480		
1926	SC4	82,320			SC3	84,970			SB6	105,680			SB5	98,460		
	SC6	73,920			SC5	87,840			SC2	97,540			SC3	104,600		
Aver.		83,190	680	1170		83,400	685	1175		103,870	850	1460		100,240	820	1410
Oct.	OA6	85,040			OA7	91,040			OA2	104,820	*		OA5	108,600	*	
27,	OB6	92,080			OB5	95,460			OB4	117,000	*		OB3	99,900	*	
1926	OC2	96,860			OB9	101,560			OC4	92,160	*		OB7	94,700	*	
	OC8	107,920			OC3	87,760			OC6	95,220	*		OC5	100,560	*	
Aver.		95,480	785	1345		93,960	770	1325		102,300	840	1440		100,940	820	1410
Grand average			740	1270			720	1240			820	1410			800	1370
High (Nov.)			845	1455		(July)	810	1390		(July)	1015	1750		(July)	990	1700
Low (Apr.)			635	1090		Apr.)	610	1050		(Mar.)	700	1200		(Mar.)	715	1230
Difference			210				200				315				275	
Difference, per cent of Grand average			28				28				38				34	

* These blocks were tested at age of 60 days.

TABLE V—COMPRESSIVE STRENGTH OF BLOCK STORED IN LABORATORY

Compression tests of 8 × 8 × 16-in. 3-core building block.
Mix 1:7 by volume.
Block cured first 24 hr. in moist room (temperature about 70° F.) or in steam room (temperature 100° to 125° F.) at plant, then stored in air of laboratory until test.
Values in parentheses are ratios expressed as percentages of compressive strengths to the strengths of companion block (Table 4) cured outdoors during the same period.
For further details of tests see notes accompanying Table 4.

Date Made	28-Day Tests								90-Day Tests							
	Initial Curing								Initial Curing							
	24 hr. in Moist Room				24 hr. in Steam Room				24 hr. in Moist Room				24 hr. in Steam Room			
	Reference No.	Breaking Load, lb.	Compressive Strength, lb. per sq. in.		Reference No.	Breaking Load, lb.	Compressive Strength, lb. per sq. in.		Reference No.	Breaking Load, lb.	Compressive Strength, lb. per sq. in.		Reference No.	Breaking Load, lb.	Compressive Strength, lb. per sq. in.	
			Gross Area	Net Area			Gross Area	Net Area			Gross Area	Net Area			Gross Area	Net Area
Dec.	DD6	98,640			DD5	85,640			DD4	87,940			DD3	88,920		
19,	DE7	92,980			DE3	94,980			DD8	103,100			DD7	98,500		
1925	DE6	108,480			DF3	96,430			DE6	99,900			DE5	113,660		
	DF4	90,900			DF5	87,370			DF8	111,600			DF7	95,900		
Aver.		97,750	800	1375 (110)		91,110	745	1285 (113)		100,640	824	1420 (105)		99,250	815	1400 (108)
Aug.	AD2	119,500			AD5	97,320			AD4	119,060			AD3	102,080		
26,	AE6	95,920			AD7	103,720			AE4	104,350			AD5	88,740		
1926	AE8	94,700			AE5	88,720			AE2	94,420			AE3	126,320		
	AF4	97,960			AF3	98,500			AF6	129,240			AF5	123,000		
Aver.		102,020	835	1435 (105)		97,060	795	1365 (104)		111,770	915	1575 (98)		110,040	900	1550 (107)
Oct.	OD2	118,300			OD5	95,040			OD4	105,480	*		OD3	118,000	*	
27,	OE6	96,400			OF3	110,030			OD6	97,370	*		OE5	87,820	*	
1926	OF8	98,480			OF3	105,420			OE4	108,420	*		OE7	91,480	*	
	OF4	94,560			OF7	93,470			OF2	98,880	*		OF5	99,620	*	
Aver.		101,940	835	1435 (107)		100,990	825	1420 (107)		102,540	840	1445 (101)		99,230	815	1400 (99)
Grand aver. .			825	1420			785	1350			860	1480			845	1450

* These block were tested at age of 60 days.

The principal data of the tests are plotted in Fig. 1.

The strengths of the four block tested for a given condition were quite uniform. In general, the individual strengths were within 10 per cent of the average strength of the four block in a set and in most instances they were within 5 per cent.

TABLE VI—SUMMARY OF METEOROLOGICAL CONDITIONS

Data are for period of outdoor curing of block and are based on reports of the Minneapolis and St. Pau Stations of the U. S. Weather Bureau. (See Fig. 1.)

Item		Date of Making Block											
		Nov. 18, 1925	Dec. 19, 1925	Jan. 18, 1926	Feb. 20, 1926	Mar. 21, 1926	Apr. 21, 1926	May 21, 1926	June 23, 1926	July 25, 1926	Aug. 26, 1926	Sept. 26, 1926	Oct. 27, 1926
28-DAY TESTS													
Average Mean Temperature, deg. F		26.3	15.4	15.0	22.4	34.5	57.7	65.0	71.4	69.2	63.4	51.0	31.1
Average Relative Humidity, per cent	7 A. M.	80.8	85.9	85.5	79.5	73.8	64.6	75.0	76.5	87.8	82.5	88.0	80.5
	Noon	63.8	71.6	68.9	59.5	42.2	35.9	47.1	46.2	65.0	65.2	58.3	65.1
	7 P. M.	64.8	78.6	74.7	64.7	46.6	34.9	46.3	44.1	64.6	69.6	62.4	71.1
	Grand Aver.	69.8	78.7	76.4	67.9	54.2	45.1	56.1	55.6	72.4	72.4	69.6	72.2
Average Absolute Humidity, 0.0001 lb. water per cu. ft. air	7 A. M.	1.76	1.21	1.29	1.34	1.97	3.96	6.20	7.68	8.30	6.45	4.05	2.23
	Noon	1.77	1.38	1.36	1.42	1.80	3.32	5.41	6.65	7.85	7.03	4.06	2.16
	7 P. M.	1.73	1.45	1.44	1.43	1.81	3.24	5.61	6.65	8.05	7.03	4.40	2.29
	Grand Aver.	1.75	1.33	1.36	1.39	1.86	3.51	5.74	6.99	8.06	6.84	4.17	2.23
Total Precipitation, inches		0.96	0.43	0.72	1.17	0.86	0.77	2.34	1.24	3.84	4.95	1.70	1.57
Total Hours Sunshine		88	86	104	180	206	274	273	301	156	130	117	95
90-DAY TESTS													
Average Mean Temperature, deg. F		19.1	18.1	24.3	39.8	54.0	63.7	69.2	69.1	61.5	48.0	32.3	23.1
Average Relative Humidity, per cent	7 A. M.	80.8	83.1	79.5	74.0	72.2	72.5	80.0	82.3	85.3	83.6	84.6	83.4
	Noon	69.3	66.9	58.0	45.0	41.7	44.0	53.0	58.3	61.6	61.5	68.0	74.5
	7 P. M.	73.2	72.0	62.7	47.6	42.0	43.6	52.0	59.0	64.1	66.1	71.5	77.6
	Grand Aver.	74.4	74.0	66.4	55.5	51.9	53.4	61.3	66.5	70.3	70.4	74.7	78.5
Average Absolute Humidity, 0.0001 lb. water per cu. ft. air	7 A. M.	1.44	1.34	1.58	2.54	4.05	5.76	7.34	7.60	6.39	4.28	2.61	1.80
	Noon	1.51	1.43	1.61	2.25	3.41	4.09	6.51	7.32	6.39	4.30	2.54	1.85
	7 P. M.	1.56	1.51	1.66	2.29	3.49	5.09	6.71	7.42	6.50	4.38	2.64	1.88
	Grand Aver.	1.50	1.43	1.62	2.38	3.65	5.18	6.85	7.45	6.43	4.32	2.59	1.84
Total Precipitation, inches		2.11	2.38	2.91	2.89	4.44	6.31	9.53	12.17	10.89	8.31	4.68	3.35
Total Hours Sunshine		3.14	4.33	5.72	7.90	8.77	9.55	8.39	6.87	5.08	3.94	2.94	1.64

There was little difference in the 28- or 90-day strength of block cured the first 24 hr. in moist room at 70 deg. F. as compared with that of block cured the first 24 hr. in steam room at a temperature of from 100 to 125 deg. F.; block cured in moist room the first 24 hr. averaged 3 per cent stronger at 28 and 90 days than block cured in the steam room the first 24 hr.

In general, the lowest strengths were obtained from block made during the winter and spring months and the highest strengths from block made during the summer and fall months.

The widest range in strength for the 12 months, based on the average of the gross area strengths for the two initial curing conditions, was 205 lb. per sq. in. at 28 days and 295 lb. per sq. in. at 90 days. These ranges are 28 and 36 per cent, respectively, of the grand average of all tests at each age.

For both initial curing conditions the strength of the block at 90 days was 11 per cent greater than at 28 days.

A comparison of the compressive strengths obtained each month with the meteorological conditions during outdoor curing shows that temperature, humidity and precipitation exerted the greatest influence on strength; the greatest strengths were generally obtained for the months having the highest temperatures and during which the air contained the most water vapor and rainfall was greatest.

Blocks cured indoors in laboratory after initial curing at plant (Table V) gave 8 per cent higher strength at 28 days and 3 per cent higher strength at 90 days than companion blocks cured outdoors after the initial curing period.

Detailed data of absorption tests at 28 days on 24 blocks are not included. The average absorption at 28 days was 5.8 per cent by weight; the difference in absorption due to the 2 different initial curing conditions or the time of the year was not appreciable.

CONCLUSIONS.

(1) There was no appreciable difference in strength at 28 or 90 days of block cured the first 24 hr. in moist room at 70 deg. F. or in steam room at 125 deg. F.

(2) The maximum range in compressive strength of the 28-day block for the 12 months was 28 per cent of the grand average of all 28-day block. For the 90-day block this range was 36 per cent.

(3) The compressive strength at 90 days averaged 11 per cent greater than that at 28 days.

(4) The summer and fall months are more favorable for outdoor curing of block than the spring and winter months.

(5) Temperature, humidity and precipitation are the most important factors entering into the curing of concrete block. In general the warm, humid and rainy seasons are preferred to secure the highest strength.

(6) The initial curing condition or season of year had no appreciable effect on the absorption of the block; the average absorption was 5.8 per cent.

APPENDIX 2.

Plant No._____

Labor Analysis in Manufacture of Concrete Block or Tile.

(Committee P-6, American Concrete Institute)

Name of Company_____

Location of Plant_____

Report by_____ Address_____

This sheet is to be detached from report when given to the committee, the members of which will recognize report by the number assigned each plant. The Secretary will file this sheet and keep the numbers assigned to each plant confidential. Correspondence regarding details of any report will be handled through the Secretary.

_____

Type of Product Manufactured_____
Block, Brick or Tile

_____ _____ _____
Size Actual Dimensions Make of Machine Average Weight

Send completed analysis to:

C. L. Bourne
Secretary Committee P-6
American Concrete Institute
33 West Grand Avenue
Chicago, Illinois

Plant No._____

LABOR ANALYSIS

CONCRETE BLOCK OR TILE.

(Committee P-6, American Concrete Institute)

1. HANDLING MATERIAL TO STORAGE.

1. Cement
   Delivered by
   How handled
   Average distance to storage place
   How stored

Labor unloading car or truck No. barrels_____ _____hrs.

@_____per hr. Total_____.

Labor unloading per barrel. .....hrs. @.....per hr.

Total.....

Cost per 1000 concrete units. .....hrs. @.....per hr.

Total.....

2. Sand, Kind..... Proportions Used..... .....%.

Washed or Pit-run

Delivered by
How handled
Average distance to storage place
How stored
Labor unloading—cubic yards. .....hrs. @.....per hr.

Total.....

Labor unloading—per cubic yard. .....hrs. @.....per hr.

Total.....

Cost per 1000 concrete units. .....hrs. @.....per hr.

Total.....

3. Coarse Aggregate, Kind..... Proportions Used.....%.

Delivered by
How handled
Average distance to storage place
How stored
Labor unloading—cubic yards. .....hrs. @.....per hr.

Total.....

Labor unloading—per cubic yard. .....hrs. @.....per hr.

Total.....

Cost per 1000 concrete units. .....hrs. @.....per hr.

Total.....

Other Materials (Calcium Chloride, Lime, etc.)

Name any other materials used
How handled
Average labor per week. .....hrs. @.....per hr.

Total.....

Average labor per hour operating. .....hrs. @.....per hr.

Total.....

Average labor per 1000 concrete units. .....hrs. @.....per hr.

Total.....

2. MIXING.

1. Describe equipment for measuring materials. State size, type and name of mixer. Size of batch and proportions.

2. Describe movements of man when measuring materials.

3. How far are materials from mixer. Cement _____ feet
   Sand _____ "
   Coarse Agg. _____ "
   Water _____ "

4. Labor hours per day mixing @_____ per hr. Total_____.

5. Cost of labor per 1000 units. _____ hrs. @_____ per hr.
   Total_____.

6. Does mixer labor handle any other work. If so, state what is done and amount of time used.

3. MAKING UNITS.

1. Describe machine. Type_____. No. of units made at one time_____. No. of men at machine and their movements in making units_____.

   What is best production obtainable per minute_____.

   What is average production per minute_____.

2. Labor hours daily operating machine. _____ hrs. @_____ per hr.
   Total_____.

   Average output per day_____.

   Cost of labor operating machine per 1000 units. _____ hrs.
   @_____ per hr. Total._____.

3. Describe off-bearing equipment in detail.

   Capacity of single off-bearing equipment_____ units.

   No. men off-bearing and their movements.

   Labor hours daily off-bearing_____ @_____ per hr.
   Total_____.

   Cost of labor off-bearing per 1000 units. _____ hrs. @_____ per hr.
   Total_____.

4. HANDLING UNITS.

(Machine to Curing Room)

1. Describe in detail operation of moving units from machine to curing room.

2. No. men_____ and labor hours daily taking units from machine to curing room_____ @_____ per hr. Total_____.

3. Cost of labor per 1000 units. _____ hrs. @_____ per hr. Total_____.

4. Is machine ever held up because off-bearing equipment is away.

   If so, how much time is lost daily_____.

   Average cost of such hold-ups per 1000 units. _____ hrs. @_____ per hr. Total_____.

5. YARD OPERATIONS.

1. Describe equipment and movements in taking units from curing room and stocking in yard_____.

2. No. men_____ and labor hrs. daily required_____ @_____ per hr. Total_____.

3. Cost of labor per 1000 units. _____ hrs. @_____ per hr. Total_____.

4. Describe movements and equipment used in moving units from stock pile to truck or car.

5. Labor hours daily required_____ @_____ per hr. Total_____.

6. Cost of labor per 1000 units. _____ hrs. @_____ per hr. Total_____.

6. MISCELLANEOUS.

1. Care of Equipment.

   (a) Unloading equipment, what done_____ labor hrs. daily_____ @_____ per hr. Total_____.

   (b) Mixer, what done_____ labor hrs. daily_____ @_____ per hr. Total_____.

(c) Products machine, what done_____ labor hrs. daily_____

@_____ per hr. Total_____.

(d) Pallets, what done_____ labor hrs. daily_____

@_____ per hr. Total_____.

(e) Off-bearing and yard equip., what done_____ labor hrs. daily_____

@_____ per hr. Total_____.

Total_____ hrs. @_____ per hr. Total_____.

Cost per 1000 units.

_____ hrs. @_____ per hr. Total_____.

2. Other labor.

State all regular jobs not previously covered.

Average labor hrs. daily_____ @_____ per hr. Total_____.

Cost per 1000 units. _____ hrs. @_____ per hr. Total_____.

3. Hold-ups and delays.

Keep record of all hold-ups and delays (even of very brief duration) for one month. For each delay note the following items. (Keep on separate sheets and attach to this report.)

(a) Cause_____.

(b) How remedied_____.

(c) Hours labor_____ @_____ per hour needed to remedy.

Total_____.

(d) Hours labor_____ @_____ per hour lost by men not working because of delay. Total_____.

(e) Materials and cost of materials needed to remedy delay_____.

Monthly cost of delays. _____ hrs. @_____ per hr.

Total_____. _____ Materials.

Cost of delays per 1000 units. _____ hrs. @_____ per hr.

Total_____. _____ Materials.

4. Over-head wages in plant. State number of men @_____ per mo.

Cost per 1000 units_____.

LABOR RECAPITULATION.

	Labor Hrs. 1,000 Units			Cost per 1,000 Units
Over-head		@	per hr.	
Handling Material to Storage		@	per hr.	
Mixing		@	per hr.	
Making Units		@	per hr.	
Moving to Curing Room		@	per hr.	
Yard Operations		@	per hr.	
Miscellaneous Care Equipment.	hrs.	@	per hr.	
Labor not otherwise noted.	hrs.	@	per hr.	
Labor cost of delays.	hrs.	@	per hr.	
		Total		
		Grand Total		

DISCUSSION.

IRA H. WOOLSON.—Were these blocks which were stored for 90 days stored out in the open? Mr. Woolson.

MR. MCKAY.—Out in the open. Mr. McKay.

MR. WOOLSON.—In piles? Mr. Woolson.

MR. MCKAY.—In piles of six high and with another block topping them on its side so that the air cores, being vertical, were covered over. Mr. McKay.

MR. WOOLSON.—Did you take any thought of whether the outside block, the block that composed the top, or the block on the side of the pile, showed any material difference from those which would be in the center of the pile, such as would be the condition under large yard storage? Mr. Woolson.

MR. MCKAY.—We were unable to do that for the reason that there were only some 30 blocks made each month. When you store 30 blocks by that method, they are all outside blocks, so that we did not have the opportunity of getting mass blocks over a large area and studying the outside as compared to the inside. Mr. McKay.

Steel Forms for Flat Slab Construction.

Submitted by Committee C-4.

During the past year Committee C-4 has been studying the use of steel forms for flat-slab construction. The following report has been compiled with the hope that it will aid in answering the following pertinent questions:

1. Is there any necessity of substituting a steel form for wood?
2. What is there on the market today to take the place of wood?
3. What about the cost of using steel forms?
4. What about the future?

Formwork is a major portion of building construction, varying from 20 to 40 per cent of the cost of that part of the work usually done by the general contractor. Forms are used to mold concrete into the structural shapes and architectural designs as shown on the building plans and called for by the specifications. They are not a part of the structure and as the building nears completion they lose their identity as one of the largest if not the largest item entering into the project.

There as numerous questions that the building constructor has probably asked at one time or another. One of these is: How can I eliminate the waste of lumber that has been used for molding concrete? A few years ago wood was used almost exclusively for concrete forms. One of the first things the general contractor does after he has secured the contract for a structure is to make up a bill of material of lumber for concrete forms. It is purchased and delivered to either the building site, his warehouse or shop, where the lumber is made up into panels. If this is done on the building site a temporary shop is installed. The detail sheets for making forms are usually made up in the general contractor's office, either in a special department or the engineering department; some organizations do the detailing of these panels on the job. The panels are used throughout the job and upon its completion the floor panels might be shipped to another job and re-used once or twice for wall forms. After this using there is very little, if any, salvage value left in them. In erection on this job they are cut to shorter lengths, and to fit around wall bracing and torn apart in removing. There is not much salvage value in the other panels. They are scrapped or torn apart; the 2-in. material re-used as battens or sawed into wedges and the 1-in. sheeting used for shore ties or other miscellaneous uses on another job.

An early construction development was the round interior column. The round metal column form was developed to simplify the erection of these

forms. Four by four's were originally used for shores or posts. To simplify the erection and reduce costs, the adjustable shore was developed. Due to the tremendous waste of lumber and small amount of salvage the construction interests will eventually require some substitute for lumber for concrete forms. It is generally conceded that there is a diminution in the supply of lumber and an inferior quality is marketed for use in concrete construction.

There are several types of steel forms for flat-slab construction on the market today. A brief description of three of these types follows: One type consists of metal panels made up in sizes of 1 ft. and 3 ft. wide x 8 ft. long, reinforced with a 1½-in. angle around the edge, further reinforced with two longitudinal wood cleats. The metal is fastened to the angle frame with countersunk rivets. These metal panels are supported by 4 in. x 6-in. stringers placed 4 ft. on centers. These stringers are supported by shores, the spacing depending upon the thickness of the slab. Before pouring, reshores are placed under the narrow panels.

Another type consists of panels of the following sizes: 1 ft. x 4 ft.; 1 ft. x 6 ft.; 2 ft. x 4 ft.; 2 ft. x 6 ft.; 1½ ft. x 4 ft.; 1½ ft. x 6 ft.; the panels 1 ft. wide being used for reshoring and the other two widths being removable, without disturbing the shoring system. The removable panels are reinforced laterally with pressed steel channels. The reshoring panels are reinforced on the ends with a light gage steel Z-bar. The panels are carried on two 2 x 8 ledgers supported by 4 x 4 shores wedged up from the floor, one 2 x 8 on each side of the 4 x 4. The reshore panel is carried on the top of the 4 x 4 shore. A cleat is nailed on each side of the shore at such a height that the 2 x 8 ledgers may be wedged up from the cleat to support the floor panels. A wood button is bolted through these cleats to prevent the ledger from turning. In stripping, the cleat wedges are knocked out and the wood button is turned 90 deg. This allows the 2 x 8 ledgers and panels to be dropped without disturbing the shores.

Another type consists of panels 1 ft. 10 in. and 2 ft. 4 in. wide x 3 ft. and 4 ft. long. The longer edges of the panel are bent over to form a bearing. The panels are reinforced laterally with strips held in place under the turned edges. These panels can be end-lapped 6 in. or less. The panels are carried on 1 x 2 strips nailed 1⅝ in. from the top edge of either the 2 x 6 or 2 x 8 joist. These joists are carried on 4 x 6-in. stringers supported on shores. In stripping, the 1 x 2-in. strips are removed from both sides of the joists releasing the panels; the joist remains for a shore.

In stripping steel forms, care must be taken to avoid dropping the panels on account of the possible damage to them as well as to the monolithic floor finish, if there is any.

The weight of steel panels is approximately 5 or 6 lb. per sq. ft. They are transported from one locality to another in box cars. Approximately 15,000 to 20,000 sq. ft. can be carried in a car, the panels being piled loose. The cost of transportation is around 0.06¢ per lb. per mile in carload lots.

The usual practice for the use of steel forms on a job is on a sub-contract basis: the sub-contractor furnishing steel panels, drop heads, lumber required to fill in odd places, all labor to place and strip material furnished by him as well as labor in placing and removing mud sills, shores, reshores, stringers and bracing; the general contractor furnishing at the site mud sills, shores, reshores, stringers and bracing in an amount agreed upon before signing contract. The general contractor furnishes and erects the wood column forms and any beams or lintels.

In using steel forms no elasticity of adjustment is obtainable. Where an under-run occurs it must be filled out with wood, where an over-run occurs certain steel sections must be left out and the opening filled in with wood. Any irregular shapes or special conditions must be formed with wood. Inserts and pipe sleeves are fastened with a marine glue or an asphaltic compound or by drilling holes and bolting or wiring to the slab form. Electric conduits are either wired, bolted, or special screws used to fasten into the metal.

There is a wide difference of opinion as to the surface condition of the concrete when steel forms are used. The marks of the joints of the panels are apparent in the finished concrete. This is also true of wood panels. The use of steel forms should not in any way produce an inferior surface. If the panels are carefully handled and properly erected a surface appearance somewhat superior to that obtained by the use of wood forms ought to result.

There have been several attempts to make steel forms for beams and girders but none of them has been economical or satisfactory. There seems to be no way in which steel forms can be adapted to the varying requirements of beam and girder work. Circular column forms, owing to the case of adjustment and resulting economies, are standard equipment. There is a steel column form on the market used for casting rectangular columns. These are small units 2 in. and 4 in. wide, of 3 ft., 4 ft. and 5-ft. lengths. Panels are made of these units with a wood strip at the edge fastened together by wood battens. These panels are clamped together with clamps similar to those used on wood column forms.

Costs.—It is difficult to make a broad statement concerning the relative costs of wood and steel forms for flat-slab construction. There are many factors affecting the cost of erection of formwork. The steel forms are usually leased for individual jobs and not purchased by the general contractor. The lease price is governed by the amount of equipment furnished and the total area of flat-slab forms. The lease price is also dependent upon the location of the job and the transportation charges. It will vary from 7¢ to 12¢ per sq. ft. The cost of erecting supporting members will vary from 3¢ to 6¢ per sq. ft. Placing the steel panels from 1¢ to 2½¢ per sq ft. The cost of installing the wood fillers necessary to fill any places where the steel panels are not adapted will vary from 10¢ per sq. ft. up. Reshores will cost from ½¢ to 1¢ per sq. ft. of floor forms. Stripping costs from 2¢ to 4¢ per sq. ft. for supporting members and 1¢ to 2¢ for

panels. Cost of cleaning and oiling for re-use will run from ½¢ to 2¢ per sq. ft. The total labor cost is therefore between 7½¢ and 17½¢ per sq. ft. adding to this the lease price, the total cost of using steel forms will vary from 14½¢ to 29½¢ per sq. ft. There were prices of 14¢ to 30¢ per sq. ft. brought to the attention of the committee. An attempt was made to determine the cost of using wood forms, basing the cost on the same operations as are included in the steel form costs. These figures varied from 12½¢ to 18¢ per sq. ft.

Conclusion.—The committee feels, after reviewing all of the data submitted, that there must at least be 6 or 8 re-uses of steel forms before they can compete with wood forms. This should be amplified in that the job should be sufficiently uniform to permit the use of steel forms without a great deal of special work. The use of steel forms is just in its infancy and further development is anticipated. The present custom of the general contractor sub-letting part of the formwork to a sub-contractor for steel forms isn't altogether satisfactory. In sub-letting the decking alone the most satisfactory part of flat-slab construction is turned over to another organization and the general contractor's costs in erecting the remaining part of the work is usually very high. It is felt that a more satisfactory arrangement might be worked out between the general contractor and the sub-contractor erecting steel forms. The sub-contractor for steel forms has an organization that is thoroughly experienced in erection. This organization is moved from job to job. On the other hand the general contractor has not had an opportunity for his organizations to become familiar with the erection of steel forms and naturally does not like to attempt the erection of a system of formwork with which he is not familiar. If he could sublet all the formwork he would be in a better position to make an intelligent decision whether to use wood or steel and it would be a matter of comparing cost records with given quotations.

In the further development of steel forms, with the idea of eliminating some of the lumber waste, might it be suggested that the supporting members be developed of steel sections. Roughly speaking the decking amounts to about one-third of the total lumber used in formwork. The adjustable shore has proven itself economical. Cannot the other supporting members be developed with sufficient elasticity of adjustment to permit their reusing a sufficient number of times, so that the unit cost chargeable on one job would compare favorably with the present use of wood members? While the committee feels that the steel form of today for flat-slab construction is not wholly satisfactory, there are future possibilities of development and efforts along this line that should not be dropped but vigorously carried on so that in the very near future the use of steel forms will greatly increase.

E. C. HARDING, *Chairman.*

DISCUSSION.

Mr. Rockwood.

E. F. ROCKWOOD.—In our minds one of the strongest arguments today for steel forms for flat slabs is the appearance of the ceiling as produced by their use. We know of a number of instances where the architect prepared plans for a steel frame office building and when called for bids they all ran too high. He then submitted an alternate of flat slab construction and specified our steel forms and omitted the plastering on the ceilings. This reduced the cost and the job went ahead. We feel, therefore, that the use of steel forms broadens the field for flat slab construction and makes it adaptable for office buildings and saves the cost of plastered ceilings in buildings so constructed.

Another advantage is the saving in labor which was brought out by the preceding speaker.

Also, in New England and other similar localities, we find that in winter less heat is required if steel forms are used because they are such excellent conductors of heat. On one steel form job we started out with five heat holes per bay, but after the job had been running a short time, the number was reduced to one per bay.

Economic Advantages, Field Control of Quality of Concrete.

Submitted by Committee C-6 on Field Methods.

The committee is charged with the task of studying methods for the field control of concrete. Its members have labored faithfully to this end, have met and discussed the subject at length and in detail. They are in agreement as to what "field control of concrete" is and how it may be obtained. But in preparing a report, they are confronted with this difficulty—the subject has already been discussed ably and fully in a number of papers before the Institute (some prepared by members of the committee), and to retrace the ground covered by these papers does not seem to the committee to be the best use of the space allotted to it for its report.

In the papers on field control so far presented, the Institute has been told why field control of concrete is advisable, how it has been applied under different circumstances and by different people, how simple it is and how a concrete of improved quality has resulted, but little has been said to convince the engineer or contractor that the methods whereby field control of concrete is accomplished are of monetary benefit to him. This is the message that the committee wishes to impart—contractors and engineers can not afford to ignore the possibilities for profit that can accrue through the application of the methods of field control and that result from attention to those very features known to be essential to quality concrete.

Different people have different conceptions of what is meant by "field control of concrete." The committee believes that it should be interpreted broadly to include the supervision of all the processes of concrete manufacture, that is, the supervision of:

(a) The quality of the constituent materials, i. e., the inspection necessary to insure a supply of cement, aggregates and water of the required quality.

(b) Design of the mixture: i. e., the determination of the relative proportions (or quantities) of the constituent materials to give concrete of the desired quality.

(c) Measurement and mixing of the constituent materials.

(d) Placing and curing of the concrete.

(e) Taking of the necessary field tests.

In this report, however, the committee is only considering the second and more especially the importance from the economic standpoint of a careful consideration of the design of the concrete mixtures.

The problem of the design of concrete mixtures divides itself into two parts, designing for quality and designing for economy. A design to be truly successful should give both the required quality and maximum economy but as in structural design, quality or ability to meet the demands made upon it in service can be obtained without reference to economy. For this reason the design of concrete for quality and for economy will be treated separately.

DESIGNING FOR QUALITY.

Investigations and experience indicate that the quality of the concrete is largely controlled by the relation between the quantity of the cement and water in the mixture, it being understood that the mixtures are workable and the constituent materials of proper quality. But it should be added that a truly workable mix can not be had without a sufficiency of fine aggregate and lacking this, the ordinary rules for the design of concrete cease to apply.

DESIGNING FOR ECONOMY.

The underlying principles in maintaining the quality of a concrete is to keep a uniform relation between the quantity of water and cement in the mixture. For any given mix, the quantity of water may be increased or decreased as required to obtain the degree of workability desired, but at the same time, cement must be added or subtracted proportionately if the quality is to be uniform. Since the cement and water must be maintained in the same proportion, designing for economy demands that a combination of materials and conditions be found that will require the use of the least mixing water to obtain the desired results.

The amount of water thus necessary is governed first by the quality and character of the cement and aggregates, secondly by the proportions in which the materials are combined and thirdly by the workability required. To this should be added another factor. That is the margin of safety that the engineer must allow in setting his mixture to provide for adverse job conditions such as variable materials, poor methods of measurement, poor workmanship, etc.

WORKABILITY AND ECONOMY.

Much has been said on the advisability of using the driest possible consistency that can be handled and placed properly. If for argument, we assume a concrete designed to have a compressive strength of 2,000 lb. per sq. in. at 28 days, using sand graded from 0 to a No. 4 sieve and

gravel from a No. 4 sieve to 1½ in., the cement required per cubic yard of finished concrete, as given by Abrams and Walker,[1] is:

Slump In.	Cement Bbl.	Aggregate—Cu Yd. Sand	Gravel
½ to 1	1.05	0.42	0.71
3 to 4	1.17	0.42	0.69
6 to 7	1.44	0.36	0.72
8 to 10	1.88	0.28	0.67

Again, if we assume that cement costs $2.30 a barrel and sand and gravel $1.45 and $1.90 a cu. yd. respectively,[2] the cost of material for these mixes becomes:

Slump In.	Cost of Material Per Cu. Yd.
½ to 1	$4.38
3 to 4	4.61
6 to 7	5.19
8 to 10	6.01

Those who have experimented in practice have found that it is possible to place, satisfactorily and economically, concretes of a consistency drier than ordinarily considered possible and that a mixture with a slump exceeding 3 to 4 in. is seldom required. If by a little forethought and planning, mixtures of a 3 to 4-in. slump can be substituted for those of a 6 to 7-in. slump, the economy resulting warrants the attention required to achieve this result.

PROPORTIONS AND ECONOMY.

Different methods have been proposed for determining the best combination of cement, aggregates and water for a given quality of concrete, but this report deals primarily with the economics of mixtures and not with their design, and the committee will not enter into a discussion of these. However, in order to exemplify the economy of a properly proportioned concrete, some method of designing a concrete mixture must be applied in the cases chosen for illustration. In this report, Abrams' fineness modulus method will be followed as it is probably best known to the majority of Institute members, but the committee does not want this choice construed as an endorsement of this method of designing mixtures, for until they have studied further this phase of their assignment, they feel it would be unwise to commit themselves in this matter.

Let us study the effect of proportions on economy by considering specific cases in which differently graded aggregates are used. Take a case

[1] "Quantities of Materials for Concrete," by Abrams and Walker, Bull. No. 9, Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill.
[2] Based on average of prices for United States as of October, 1926.

commonly met with: a mixture designed on the basis of a concrete having a compressive strength at 28 days of 2,000 lb. per sq. in. where the common practice is followed in assuming this to be a 1:2:4 concrete. Let us compare the cost of this 1:2:4 concrete with a concrete designed for the required strength. In order to make the results general, let us work out the cost of mixtures made from three differently graded sands combined with two differently graded gravels, each with each, making six different combinations. The quantities of materials required and their cost for both 1:2:4 and 2,000-lb. concrete are given in Table I. It should also be remembered that the 1:2:4 concrete in these examples may or may not have a strength of 2,000 lb. per sq. in. at 28 days, and while in the majority of cases where care is exercised in maintaining proper consistencies the

TABLE I—ARBITRARY VS. DESIGNED MIXTURES

COMPARISON OF COST OF MATERIAL REQUIRED FOR ONE CU. YD. OF CONCRETE

Grading of Sand	0 to ¼ in.		0 to ¼ in.		0 to No. 4		0 to No. 4		0 to No. 8		0 to No. 8	
Grading of Gravel	No. 4 to 1½ in.		¼ to 1½ in.		¼ to 1½ in.		¼ to 1½ in.		No. 4 to 1½ in.		¼ to 1½ in.	
Concrete designed for	1:2:4	2000 lb.	1:2:4	2000 lb.	1:2:4	2000 lb.	1:2:4	2000 lb.	1:2:4	2000 lb.	1:2:4	2000 lb.
Cement per cu. yd., bbl	1.28	1.18	1.28	1.13	1.28	1.16	1.30	1.13	1.30	1.16	1.32	1.10
Sand per cu. yd., cu. yd	0.38	0.44	0.38	0.58	0.38	0.36	0.38	0.52	0.38	0.36	0.39	0.48
Gravel per cu. yd., cu. yd	0.75	0.67	0.75	0.52	0.75	0.78	0.77	0.65	0.77	0.81	0.78	0.67
Cost of materials per cu. yd.*	$4.91	$4.62	$4.91	$4.43	$4.93	$4.67	$5.00	$4.58	$5.00	$4.73	$5.09	$4.50
Additional cost of 1:2:4	$0.29	....	0.48	....	0.26	....	0.42	....	0.27	....	0.59	....

* Costs based on the following prices for cement and aggregates.

Cement	$2.30 per bbl.
Sand	$1.45 per cu. yd.
Gravel	$1.90 per cu. yd.

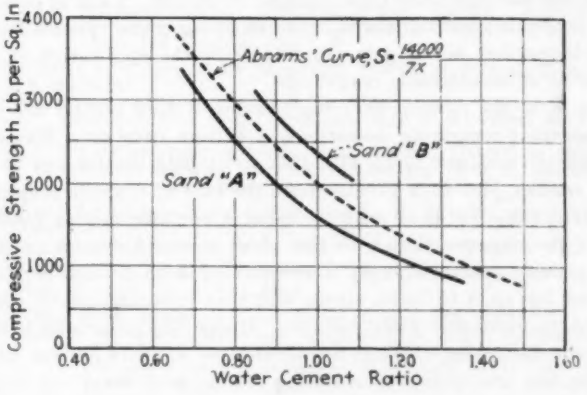
Characteristics of aggregates:		
Grading 0 to ¼ in.	F. M. 4.00	Unit weight 121 lb.
0 to No. 4	3.05	115 lb.
0 to No. 8	2.70	109 lb.
No. 4 to 1½ in.	7.00	106 lb.
¼ to 1½ in.	8.00	104 lb.

required strength would be obtained, there are many materials, which, when combined in a proportion of 1:2:4, will not give this strength. If anyone doubts this, let him study the table of proportions for 2,000-lb. concrete contained in the report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete.

An actual instance of such a saving has been reported to the committee by one of its members, J. G. Ahlers. In the case in question (a 7-story reinforced-concrete warehouse), preliminary tests showed a 1:2:4 gravel concrete would cost $6.20 per cu. yd. but using a concrete designed to have a compressive strength of 2,000 lb. per sq. in. at 28 days, the actual cost per cubic yard averaged $5.62 or a saving of 58 cents per cubic yard. As there were 1,494 cu. yds. in the job, the total saving effected was $866.52. In this case, cement cost $2.50 per barrel and aggregate $2.40 per cu. yd.

These examples serve to show the difference in cost that may exist between a concrete designed scientifically and one proportioned arbitrarily. The difference may be even more marked if such factors as placing and finishing costs could be included with these figures.

Our discussion so far has been predicated on the assumption that the relation between the water-cement ratio of a concrete and its compressive strength does not differ for different materials. The experience of Ahlers, Young and others, however, shows that the general relationship of Abrams does not always apply exactly and that sometimes considerable economies can be effected if the exact relationship is known for the materials to be used. As an illustration of what this may mean in costs, take the following case, the tests for which were made by one of the members of the



COMPARISON OF WATER-CEMENT RATIO CURVES FOR CONCRETE MADE USING DIFFERENT SANDS BUT SAME GRAVEL AND SAME BRAND OF CEMENT.

committee. Two materials, A and B, of about the same grading, were combined with the same coarse aggregate and cement in the same proportions. Fig. 1 shows the water-cement ratio curves obtained. Concretes designed to have a compressive strength at 28 days of 2,000 lb. per sq. in. with a 3 to 4-in. slump would require cement and aggregates and would cost, assuming the same unit prices used before, as follows:

Aggregate	Cement Bbls.	Sand Cu. Yd.	Gravel Cu. Yd.	Cost
A	1.15	0.41	0.68	$4.53
B	0.96	0.42	0.70	4.15

JOB CONDITIONS AND ECONOMY.

This question can only be touched upon briefly here as it is to be dealt with in one of the committee's future reports.

Abrams recommends two water-cement ratio curves, one for the average job, and the other for the job that is closely supervised. Mixtures designed for the first condition required roughly a half bag of cement more for the same classes of concrete than those designed for the second. This is, in effect, the inclusion of a factor of safety of a half bag of cement per cu. yd. to provide for a lack of care in handling and measuring materials, workmanship, etc. In other words, the penalty that Abrams recommends for laxity in field control is roughly a matter of 50 to 60 cents per cubic yard of concrete.

But there are other penalties paid by the organization that permits haphazard methods and lax supervision of their concrete. Inexact measurements often result in an overrun of 10 to 15 per cent in the cement used. With a chuting plant, they result in occasional dry batches causing plugs in the chutes and expensive interruptions in placing. Lack of care in placing and poorly designed mixes increase finishing costs. Losses from these sources (losses that are largely preventable by the application of job control) can be of considerable magnitude.

Many hold the opinion that the niceties of field control are only for the monumental concreting operations, a Wilson dam or a Pitt stadium. The committee does not agree with this view; field control can be applied to much smaller jobs with profit. Take the case of a small project with a few hundred cubic yards of concrete using a portable mixing plant of the simplest type: Assume that with this plant it would require an outlay of $100 to provide means whereby a concrete of 6 to 7-in. slump could be substituted for an 8 to 10-in. slump and that it would cost 5 cents more per cu. yd. to place this drier concrete. Under the conditions assumed in the previous discussion, there is a difference of 80 cents per cu. yd. in the cost of the two concretes after allowing for the additional cost of placing, and this would pay for the initial expenditure of $100 in 125 cu. yd.

Take another case. Suppose the testing required to design properly a concrete mixture by the fineness modulus method costs $25. Under the conditions of Table I, this would be recovered in from 40 to 100 cu. yd. of concrete.

Again consider the case where there is a difference in the concrete-making properties of the aggregates, when preliminary testing is required to establish the exact water-cement ratio curve. Such tests should not cost over $300 and this would be recovered on a job of 800 cu. yd. These examples are merely cited to show that methods of field control are not necessarily for the big job alone; the small job can benefit thereby.

A bibliography of the literature on the field control of the quality of concrete is appended to this report. This has been prepared at the committee's request by the Portland Cement Association and is published with its permission.

R. B. YOUNG, *Chairman.*

APPENDIX

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(December, 1926.)

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Results of Field Tests of Bridge Concrete, by H. D. Miller;
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Eng. News-Rec., v. 95, p. 182, July 30, 1925.
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Ten and 28-day strength tests now standard practice by California Highway Comm.
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Comparison in California building between ordinary volumetric, weight and inundation methods of proportioning.

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South Water Street Improvement, by T. A. Evans;
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Chart Presentation of Elements Affecting Concrete Construction, by C. S. Pope;
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Effect of mix, voids in coarse aggregate, amount of mixing water, time of curing, etc., on strength of concrete at 28 days.
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Eng. News-Rec., v. 96, p. 440, March 18, 1926.
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Inundation System for Constant Concrete;
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Old and New Ideas of Control of Strength of Concrete, by J. G. Ahlers;
Journal of Boston Soc. Civil Eng., v. 13, p. 253, June, 1926.
Water-cement ratio method of proportioning.
Concrete Strength Made Uniform by Careful Proportioning, by Z. Witkin;
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Various methods of proportioning concrete tried on building construction in Los Angeles; inundation method found to be most satisfactory for production of uniform concrete.

DISCUSSION.

Mr. Miner. J. L. MINER.—Wherein does Sand *B* differ from Sand *A*? I do not believe this is covered in the report.

Mr. Young. R. B. YOUNG.—I do not know. Our present knowledge does not seem to indicate what the difference between these sands is. I have had other cases such as this one; the sand in these cases are from practically the same locality, yet have the most surprising differences in their concrete-making properties. A study of their absorptive properties or a casual examination does not give any clue as to the answer to their difference.

Mr. Fatout. RAY T. FATOUT.—Have you made any studies of what, in money, is necessary to effect this control on the ordinary job? Have you any data on that?

Mr. Young. MR. YOUNG.—I do not believe the committee, as a committee, has, but individually we have information on what it would cost. Generally speaking, the engineering supervision, using the control method, does not exceed what would be required on a job of like size under ordinary methods, that is, it does not require any more supervision to put into effect methods of field control than it does properly to inspect a job not using those methods. Occasionally you come to a border-line job where to carry out your tests you have to have an extra man usually a junior, where without those tests you could manage without him. Ordinarily where you need an extra man for tests you would need a second inspector, and where one inspector is sufficient he is able to cover the whole job. I made an estimate once as to what testing supervision and expert advice would cost on the job where a consultant might be necessary and I would say from 10c. a yard down.

Reinforced Concrete Building Design and Specifications.

Submitted by Committee E-1.

This committee submitted a "Preliminary Draft of Proposed Building Regulations for the Use of Reinforced Concrete" to the convention in February, 1925. It was accepted as a progress report and published in the *Proceedings*, Vol. XXI, 1925. The committee submitted this draft at the 1927 convention unchanged with the recommendation that it be adopted by this convention as Tentative Standard Building Regulations for the Use of Reinforced Concrete.*

F. R. McMillan, *Chairman.*

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* Three amendments to this report were offered during discussion of the report on the convention floor: the amendment to include blast-furnace slag in definition and in name was adopted; the amendment to remove the limitations on sizes of high-carbon steel reinforcing was lost; and the amendment to include the A. S. T. M. Specifications on wire reinforcing as used in building construction was unanimously adopted. The report, including these two amendments as hereinafter printed, was adopted as a tentative standard.

AMERICAN CONCRETE INSTITUTE STANDARD.

Tentative Building Regulations for the Use of Reinforced Concrete.

(Reprinted from pages 427 to 460, Vol. 21, American Concrete Institute *Proceedings*.)

(Serial Designation E-1A-27T)

CHAPTER A.

Introduction.

A-1: *Scope.*—These regulations are drawn to cover the use of reinforced concrete in any structure to be erected under the provisions of the building code of which they form a part. They are intended to supplement the general provisions of the code in order to provide for the proper design and construction of structures of this material. In all matters pertaining to the design and construction where these specific regulations are in conflict with other provisions of the code, these regulations shall govern.

A-2: *Application for Permit.*—Application for permit to erect or alter a reinforced concrete structure shall, in addition to complying with the general requirements of the code, be accompanied by duplicate sets in blueprint form of drawing which shall fully show the essential structural details by plans, elevations, sections and bending diagrams. Plans shall be legibly drawn to an appropriate scale. The principal distances and dimensions shall be accurately shown in figures, and the floor loads and strength of concrete for which the structure is designed shall be clearly indicated. The grade of reinforcing steel to be used, whether plain or deformed bars, and all other information necessary to permit the commissioner of buildings or his authorized representative to determine whether the design conforms with these regulations shall be given.

Each plan and drawing shall bear the seal of the architect or engineer as required by the law in this city or state.

The application for permit shall be made by the owner or his representative and shall be accompanied by a certificate signed and sealed by the architect or engineer responsible for the design stating that the construction shown on the plans and details submitted complies with these regulations.

A-3: *Approved Drawings to be at the Work.*—One set of drawings bearing the approval of the commissioner of buildings shall be returned to the applicant when permit to erect or alter is issued. This shall be kept at the site of the construction work at all times until the acceptance of the structure. The other set approved in like manner shall be kept by the commissioner of buildings for a period of two years after the acceptance of the structure.

A-4: *Changes in Plans.*—If any changes from the approved plans, affecting the strength of the structure or the compliance with any other provision of the code are contemplated at any time, revised blueprints in duplicate showing the revised construction and details, shall be submitted to the commissioner of buildings and his approval secured before such

changes are made. The approved revised drawings shall replace those rendered obsolete both at the work and in the office of the commissioner of buildings.

A-5: *Violations of These Regulations.*—Should any portion of the construction violate these regulations or other provisions of the code, the same shall be remedied to comply with the requirements or the construction shall be rated for its safe carrying capacity as computed under these regulations.

A-6: *Acceptance of the Structure.*—Upon the completion of the structure the owner or his representative shall notify the commissioner of buildings who shall make or authorize to be made by his representative, a final inspection and shall issue a certificate of acceptance.

No structure shall be used or occupied until it has been accepted and such certificate issued. The acceptance of the structure in this manner shall not prevent the city from requiring full compliance with these regulations should any violations be subsequently discovered.

A-7: *Posting of Loads.*—The commissioner of buildings shall issue signed placards to be posted on each floor, showing the maximum safe load per square foot which may be placed on the floor. It shall be unlawful to load any such floors or any part thereof to a greater extent than the loads indicate on such placards.

CHAPTER B.

DEFINITIONS.

B-1.—The following definitions give the meaning of certain terms as used in these regulations:

Aggregate.—Inert material which is mixed with portland cement and water to produce concrete; in general, aggregate consists of sand, pebbles, gravel, crushed stone, blast-furnace slag, or similar materials.

Anchorage.—The embedment in concrete of a portion of a reinforcement bar, either straight or with hooks, designed to prevent pulling out or slipping of the bar when subjected to stress. (The anchorage of tension reinforcement in beams includes only the embedded length beyond a point of contra-flexure or of zero moment.)

Blast-Furnace Slag.—The non-metallic product consisting essentially of silicates and alumino silicates of lime which is developed simultaneously with iron in a blast furnace.

Column.—An upright compression member the length of which exceeds three times its least lateral dimension.

Column Capital.—An enlargement of the upper end of a reinforced concrete column designed and built to act as a unit with the column and flat slab.

Column Strip.—One of two strips in a flat slab panel each ¼ panel in width, occupying the two quarter panel areas outside of the middle strip. (See *Middle Strip.*)

Composite Column.—A circumferentially reinforced concrete column with a core of structural steel or cast iron which is designed to carry a portion of the load.

Concrete.—A mixture of portland cement, fine aggregate, coarse aggregate and water. (See *Mortar.*)

Consistency.—A general term used to designate the relative plasticity of freshly mixed concrete or mortar.

Dead Load.—The weight of the permanent parts of the structure.

Deformed Bar.—Reinforcement bar with shoulders, lugs, or projections formed integrally with the bar during rolling.

Diagonal Direction.—A direction parallel or approximately parallel to the diagonal of the panel of a flat slab.

Dropped Panel.—The structural portion of a flat slab which is thickened throughout an area surrounding the column capital.

Effective Area of Concrete.—The area of a section of the concrete which lies between the tension reinforcement and the compression surface in a beam or slab.

Effective Area of Reinforcement.—The area obtained by multiplying the right cross-sectional area of the metal reinforcement by the cosine of the angle between its direction and that for which the effectiveness of the reinforcement is to be determined.

Flat Slab.—A concrete slab having reinforcement bars extending in two or more directions without beams or girders to carry the load to supporting members.

Footing.—A structural unit used to distribute wall or column loads to the foundation materials.

Gravel.—Rounded particles larger than sand resulting from the natural disintegration of rocks. (See *Sand.*)

Laitance.—Extremely fine material of little or no hardness which may collect on the surface of freshly-deposited concrete or mortar, resulting from the use of excess mixing water, and usually recognized by its relatively light color.

Live Load.—Loads and forces other than the dead load.

Middle Strip.—A portion of a flat slab panel one-half panel in width, symmetrical with respect to the panel center line and extending through the panel in the direction in which moments are being considered.

Mortar.—A mixture of portland cement, fine aggregate and water. (See *Concrete.*)

Negative Reinforcement.—Reinforcement so placed as to take tensile stress due to negative bending moment.

Panel Length.—The distance in either rectangular direction between centers of two columns of a panel.

Pedestal.—An upright compression member whose height does not exceed three times its least lateral dimension.

Pedestal Footing.—A column footing projecting less than one-half its depth from the faces of the column on all sides and having a depth not more than three times its least width.

Plain Concrete.—Concrete without metal reinforcement.

Portland Cement.—The product obtained by finely pulverizing clinker

produced by calcining to incipient fusion an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.

Positive Reinforcement.—Reinforcement so placed as to take tensile stress due to positive bending moment.

Principal Design Section.—The vertical sections in a flat slab on which the moments in the rectangular directions are critical. (See Section K-2.)

Ratio of Reinforcement.—The ratio of the effective area of the reinforcement cut by a section of a beam or slab to the effective area of the concrete cut by that section.

Rectangular Direction.—A direction parallel to a side of the panel of a flat slab.

Reinforced Concrete.—Concrete in which metal is embedded in such a manner that the two materials act together in resisting forces.

Sand.—Small grains resulting from the natural disintegration of rocks. (See *Gravel.*)

Screen.—A metal plate with closely spaced circular perforations. (See *Sieve.*)

Sieve.—Woven wire cloth with square openings. (See *Screen.*)

Strut.—A compression member other than a column or pedestal.

Wall Beam.—A reinforced concrete beam which extends from column to column along the outer edge of a wall panel.

CHAPTER C.

MATERIALS AND TESTS.

C-1: *Tests.*—The tests called for in these regulations when ordered by the commissioner of buildings or his authorized representatives shall be arranged for by the owner or his representative. No responsibility for the expense of these tests shall attach to the Department of Buildings. Such tests shall be made in accordance with the standard method of test covering the particular material under consideration, of the American Society for Testing Materials in effect on the date of the adoption of these regulations.

All such tests shall be made by competent persons approved by the commissioner of buildings and copies of the results shall be kept on file in the office of the commissioner of buildings for a period of two years after the acceptance of the structure.

Tests shall be made on any material entering into concrete or reinforced concrete construction when in the opinion of the commissioner of buildings or his authorized representative, there is any doubt as to its suitability for the purpose.

The commissioner of buildings or his authorized representative shall have the right to require tests of the concrete from time to time to determine whether the materials and methods in use are such as to produce concrete of the necessary quality. Specimens for such tests shall be taken at the place where concrete is being deposited, and shall be taken, cured

and tested in accordance with the standards of the American Society for Testing materials in effect on the date of adoption of these regulations.

C-2: *Load Tests.*—The commissioner of buildings or his authorized representative shall have the right to order the test under load of any portion of a completed structure, when the conditions have been such as to leave any doubt as to the adequacy of the structure to serve the purpose for which it was intended. Such tests shall not be required to be made on any concrete construction which is less than 60 days old.

In such tests, the member or portion of the structure under consideration shall be subject to a superimposed load equal to 1½ times the live-load plus ½ of the dead-load. This load shall be left in position for a period of 24 hours before removal. If during the test, or upon removal of the load, the member or portion of structure shows signs of failure, the commissioner of buildings shall have the right to rate the structure, or such portions thereof as are of the same character as the portions tested, for live-load less than that for which it was designed.

The holder of a permit for the construction of any building which, through failure of portions to pass the test, has been rated for a live-load less than that for which it was designed, shall have the right to submit other portions of the building to test and any portions which he can show to be satisfactory for the designed load shall be exempt from the application of the reduced live-load rating. He shall also have the right to retest any portion of the structure, provided sufficient time has elapsed and proper effort been made to correct the defects. If the portions retested prove satisfactory to the commissioner of buildings, the designed live-load rating shall be restored on all portions which have been similarly corrected.

In tests applied to determine the suitability of slab or beam construction, the structure will be considered to have failed to pass the test if within 24 hours after the removal of the load the slabs or beams do not show a recovery of at least 75 per cent of the maximum deflection shown during the 24 hours while under load.

C-3: *Inspection.*—All concrete work shall be inspected by the architect or engineer responsible for its design or by a competent representative responsible to the architect or the engineer. A record shall be kept of such inspection which shall cover the quality and quantity of concrete materials, including water, the mixing and placing of the concrete, and the placing of the reinforcing steel. The inspection record shall also include a complete record of the progress of the work and of the temperatures, when these fall below 40 deg. F., and of the protection given to the concrete while curing. These records shall be available for inspection by the commissioner of buildings at all times during the progress of the work and shall be preserved for two years after the acceptance of the structure.

C-4: *Portland Cement.*—Portland cement shall conform to the Standard Specifications and Tests for Portland Cement (Serial Designation C 9-21) of the American Society for Testing Materials.

C-5: *Concrete Aggregates.*—Concrete aggregates shall consist of natural sands and gravels, crushed rock, air-cooled blast-furnace slag or other inert materials having clean, uncoated grains of strong and durable minerals and shall meet the approval of the commissioner of buildings. Aggregates containing soft, friable, thin, flaky, elongated or laminated particles totaling more than 3 per cent, or containing shale in excess of 1½ per cent, or silt and crusher dust finer than the No. 100 standard sieve in excess of 2 per cent shall not be used. These percentages shall be based on the weight of the combined aggregate as used in the concrete. When all three groups of these deleterious materials are present in the aggregates, the combined amounts shall not exceed 5 per cent by weight of the combined aggregate. Aggregates shall not contain strong alkali, or organic material which gives a color darker than the standard color when tested in accordance with the standard colorimetric test of the American Society for Testing Materials.

The maximum size of the aggregate shall be not larger than one-fifth of the narrowest dimension between forms of the member for which the concrete is to be used nor larger than three-fourths of the minimum clear spacing between reinforcing bars. By maximum size of aggregate is meant the clear space between the sides of the smallest square opening through which 95 per cent by weight of the material can be passed.

C-6: *Water.*—Water used in mixing concrete shall be clean, and free from injurious amounts of oil, acid, alkali, organic matter or other deleterious substances.

C-7: *Metal Reinforcement.*—Metal reinforcement shall conform to the requirements of the Standard Specifications for Billet Steel, Concrete Reinforcement Bars of Structural or Intermediate Grade (Serial Designation: A15-14) of the American Society for Testing Materials. Hard-grade billet steel meeting the requirements of the above specification (A 15-14) or rail steel meeting the requirements for Rail Steel Concrete Reinforcement Bars (Serial Designation A 16-14) of the American Society for Testing Materials, may be used for bars ¾ in. in size and smaller or for larger sizes where no bending is required. The provision in these specifications for machining deformed bars before testing shall be eliminated.

Wire for concrete reinforcement shall conform to the requirements of the Tentative Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement. (Serial Designation: A82-21T of the A. S. T. M.)

Metal reinforcement, to receive the rating of "deformed bars" which permits the use of higher bond stresses than for plain bars, shall show a bond strength 25 per cent greater than that shown by plain bars of equivalent cross-sectional area.*

C-8: *Storage of Materials.*—Cement and aggregates shall be stored at the work in a manner to prevent deterioration or the intrusion of foreign matter. Any material which has deteriorated or has been damaged shall be immediately and completely removed from the work.

* The committee has under consideration a test for bond which it will propose as a basis for judging the bond value of different bars.

CHAPTER D.

CONCRETE QUALITY AND PROPORTIONS.

D-1: *Concrete Quality.*—Provisions for the design of structures embodied in these regulations are based on the presumption of concrete of certain strength. To produce concrete of the required strength, the proportion of the mixing water to the cement shall be accurately controlled. To obtain the strengths indicated in the following table, the ratio of water to cement shall be in the proportions shown. The strengths indicated represent the minimum ultimate strength in compression which may be expected at 28 days when cured and tested as specified in Section C-1.*

PROPORTION OF MIXING WATER TO CEMENT.*

Ultimate Strength Used in Design lb. per sq. in.	Water-Cement Ratio U. S. Gal. of Water Per Sack of Cement.
1500	8¼
2000	7¼
2500	6½
3000	5¾

Water or moisture contained in the aggregates must be included in determining the ratio of water to cement.

All structural drawings and plans submitted for approval shall show the strength of concrete to be used and the water-cement ratio necessary to produce that strength as per this table. Such note indicating the water required shall clearly state that this quantity of water includes that contained in the aggregates.

D-2: *Concrete Proportions and Consistency.*—The proportions of aggregates to cement for concrete of any water-cement ratio shall be such as to produce concrete that will work readily into the corners and angles of the form and around the reinforcement without excessive puddling or spading and without permitting free water to collect on the surface. The combined aggregate shall be of such composition of sizes that when separated by the No. 4 standard sieve, the weight retained on the sieve shall not be less than one-half nor more than two-thirds of the total nor shall the amount of coarse material be such as to produce harshness in placing or honeycombing in the structure. When forms are removed, the faces and corners of the members shall show smooth and sound throughout.

D-3: *Control of Proportions.*—The methods of measuring concrete materials shall be such that the proportion of water to cement can be accurately controlled during the progress of the work and easily checked at any time by the commissioner of buildings or his authorized representative. A tolerance of ¼ gal. of water per sack of cement in any batch of concrete

* Attention is called to the fact that these proportions of water to cement are based on a wide range of tests and experience and therefore they may be expected to apply to a majority of cases. However, there may be localities where the available materials are such that different water-cement ratios are required for the strengths indicated. It would be advisable therefore for each municipality to conduct the necessary tests to determine the suitability of these limits. For the purpose of a building code it is recommended that the strengths specified be not over 80 per cent of the average values shown by the tests.

will be allowed provided that the average for any 10 consecutive batches does not show a water content greater than that shown in the table and on plans as specified in Section D-1.

The method of delivering the aggregates to the work and of storing and handling shall be such that the moisture content of the aggregates as they come to the mixer shall not be subject to frequent or unnecessary changes.

CHAPTER E.

MIXING AND PLACING CONCRETE.

E-1: *Mixing.*—The concrete shall be mixed until there is a uniform distribution of the materials and the mass is uniform in color and homogeneous. In machine mixing, only batch mixers shall be used. Each batch shall be mixed at least one minute after all the materials are in the mixer and must be completely discharged before recharging.

E-2: *Cleaning Forms and Equipment.*—Before placing concrete all equipment for mixing and transporting the concrete shall be cleaned, all debris shall be removed from the places to be occupied by the concrete, forms shall be thoroughly wetted (except in freezing weather) or oiled, and clay tile that will be in contact with concrete shall be well drenched (except in freezing weather). Reinforcement shall be thoroughly cleaned and secured in position. Concrete shall not be placed until the forms and reinforcement have been inspected by the architect or engineer responsible for the design.

E-3: *Removal of Water from Excavation.*—Water shall be removed from excavations before concrete is deposited, unless otherwise directed by the commissioner of buildings. Any flow of water into the excavation shall be diverted through proper side drains to a sump, or be removed by other approved methods which will avoid washing the freshly deposited concrete. Water vent pipes and drains shall be filled by grouting or otherwise, after the concrete has thoroughly hardened.

E-4: *Transporting.*—Concrete shall be handled from the mixer to the place of final deposit as rapidly as practicable by methods which shall prevent the separation or loss of the ingredients. It shall be deposited as nearly as practicable in its final position to avoid rehandling or flowing. Under no circumstances shall concrete that has partially hardened be deposited in the work.

When concrete is conveyed by chuting, the plant shall be of such size and design as to insure a practically continuous flow in the chute. The slope of the chute shall be such as to allow the concrete to flow without separation of the ingredients. The delivery end of the chute shall be as close as possible to the point of deposit. When the operation is intermittent, the spout shall discharge into a hopper. The chute shall be thoroughly flushed with water before and after each run; the water used for this purpose shall be discharged outside the forms.

E-5: *Placing.*—Concrete shall be thoroughly compacted by puddling with suitable tools. In thin walls or inaccessible portions of the forms where rodding or spading is impracticable, the concrete shall be worked

into place by tapping or hammering the forms adjacent to the freshly deposited concrete. When necessary, openings shall be provided in the forms to permit the placing of concrete in such a manner as to avoid accumulations of hardened concrete on the forms or reinforcing bars. The concrete shall be thoroughly worked around the reinforcement, around embedded fixtures, and into the corners of the forms.

E-6: *Curing.*—Exposed surfaces of concrete shall be kept moist for a period of at least 7 days after being deposited.

E-7: *Depositing in Cold Weather.*—When depositing concrete at freezing or near freezing temperatures, the concrete shall have a temperature of at least 40 deg. F., but not more than 120 deg. F. The concrete shall be maintained at a temperature of at least 50 deg. F. for not less than 72 hours after placing or until the concrete has thoroughly hardened. When necessary, concrete materials shall be heated before mixing. Dependence shall not be placed on salt or other chemicals for the prevention of freezing.

E-8: *Bonding Fresh and Hardened Concrete.*—Before depositing new concrete on or against concrete which has set, the forms shall be retightened, the surface of the set concrete shall be roughened, cleaned of foreign matter and laitance, and thoroughly wetted but not saturated. To insure excess mortar at the juncture of hardened and newly deposited concrete, the cleaned and wetted surfaces of the hardened concrete, including vertical and inclined surfaces, shall be slushed with a coating of 1: 2 cement mortar against which the new concrete shall be placed before the mortar has attained its initial set.

CHAPTER F.

FORMS AND DETAILS OF CONSTRUCTION.

F-1: *Design of Forms.*—Forms shall conform to the shape, lines and dimensions of the member as called for on the plans.

Forms shall be substantial and sufficiently tight to prevent leakage of mortar; they shall be properly braced or tied together so as to maintain position and shape and insure safety to workmen and passersby. If adequate foundation for shores cannot be secured, trussed supports shall be provided.

Temporary openings shall be provided at the base of column and wall forms, and at other points where necessary, to facilitate cleaning and inspection immediately before depositing concrete.

F-2: *Removal of Forms.*—Forms shall not be disturbed until the concrete has hardened sufficiently to permit their removal with safety. Shoring shall not be removed until the member has acquired sufficient strength to safely support its weight and the load upon it. Members subject to additional loads during construction shall be adequately shored to support both the member and construction loads in such a manner as will protect the member from damage by the loads.

F-3: *Cleaning and Bending Reinforcement.*—Metal reinforcement, before being placed, shall be thoroughly cleaned of loose mill and rust scale

and of other coatings that will destroy or reduce the bond. Reinforcement shall be carefully formed to the dimensions indicated on the plans. Cold bends shall be made around a pin having a diameter of four or more times the least dimension of the bar.

Metal reinforcement shall not be bent or straightened in a manner that will injure the material. Bars with kinks or bends not shown on the plans shall not be used. Heating of reinforcement will be permitted only when approved by the commissioner of buildings.

F-4: *Placing Reinforcement.*—Metal reinforcement shall be accurately placed and secured, and shall be supported by concrete or metal chairs or spacers, or metal hangers. The minimum clear distance between parallel bars shall be 1½ times the diameter for round bars or 1½ times the diagonal for square bars; if the ends of bars are anchored as specified in Sec. J-5, the clear spacing may be made equal to the diameter of round bars or to the diagonal of square bars, but in no case shall the spacing between bars be less than 1 in., nor less than 1 1/3 times the maximum size of the coarse aggregate. Bars at the upper face of any member shall be embedded a clear distance of not less than one diameter.

F-5: *Splices and Offsets in Reinforcement.*—In slabs, beams and girders, splices of reinforcement shall not be made at points of maximum stress without the approval of the commissioner of buildings. Splices, where permitted, shall provide sufficient lap to transfer the stress between bars by bond and shear. In such splices the bars shall be spaced at the minimum distance specified in Sec. F-4.

Splices in columns, piers and struts shall provide sufficient lap to transfer the stress by bond.

Where changes in the cross-section of a compression member occur, the longitudinal bars shall be sloped for the full length of the member or offset in a region where lateral support is afforded. Where offset, the slope of the inclined portion from the axis of the member shall not be more than 1 in 6.

F-6: *Protective Covering of Concrete.*—At the under side of footings metal reinforcement shall have a minimum covering of 3 in. of concrete. At other surfaces of concrete exposed to the ground or weather, metal reinforcement shall be protected by not less than 2 in. of concrete.

In fire-resistive construction, metal reinforcement shall be protected by not less than 1 in. of concrete in slabs and walls, and not less than 1½ in. in beams, girders and columns, provided aggregate showing an expansion not materially greater than that of limestone or trap rock is used; when impracticable to obtain aggregate of this grade, the protective covering shall be ½ in. thicker and shall be reinforced with metal mesh having openings not exceeding 3 in., placed 1 in. from the finished surface.

In structures where the fire hazard is limited, the metal reinforcement shall not be placed nearer the exposed surface than ¾ in. in slabs and walls or 1½ in. in beams, girders and columns.

Exposed reinforcement bars intended for bonding with future extensions shall be protected from corrosion.

F-7: *Construction Joints.*—Joints not indicated on the plans shall be so made and located as to least impair the strength of the structure. Where a horizontal joint is to be made, any excess water and laitance shall be removed from the surface after concrete is deposited. Before depositing of concrete is resumed the hardened surface shall be treated as specified in Sec. E-8. Column forms shall be filled to a depth of 4 in. with 1:2 cement mortar immediately before depositing concrete.

At least two hours must elapse after depositing concrete in the columns or walls before depositing in beams, girders, or slabs supported thereon. Haunches and column capitals shall be considered as part of and to act continuous with the floor.

Construction joints in floors shall be located near the middle of spans of slabs, beams or girders, unless a beam intersects a girder at this point, in which case the joints in the girders shall be offset a distance equal to twice the width of the beam. Provision shall be made for shear by use of inclined reinforcement.

CHAPTER G.

DESIGN—GENERAL CONSIDERATIONS.

G-1: *Assumptions.*—The design of reinforced-concrete members under these specifications shall be based on the following assumptions:

(a) Calculations are made with reference to working stresses and safe loads rather than with reference to ultimate strength and ultimate loads.

(b) A plane section before bending remains plane after bending, shearing distortions being neglected.

(c) The modulus of elasticity of concrete in compression is constant within the limits of working stresses and the distribution of compressive stress in beams is rectilinear.

(d) The moduli of elasticity of concrete in computations for the position of the neutral axis, for the resisting moment of beams, and for compression of concrete in columns, are as follows:

(1) $\frac{1}{15}$ that of steel, when the compressive strength of the concrete at 28 days exceeds 1,500 and does not exceed 2,200 lb. per sq. in.;

(2) $\frac{1}{12}$ that of steel, when the compressive strength of the concrete at 28 days exceeds 2,200 and does not exceed 2,900 lb. per sq. in.;

(3) $\frac{1}{10}$ that of steel, when the compressive strength of the concrete at 28 days is greater than 2,900 lb. per sq. in.

(e) In calculating the moment of resistance of reinforced-concrete beams and slabs the tensile resistance of the concrete is neglected.

(f) The bond between the concrete and the metal reinforcement remains unbroken throughout the range of working stresses. Under compression the two materials are therefore stressed in proportion of their moduli of elasticity.

(g) Initial stress in the reinforcement due to contraction or expansion of the concrete is neglected, except in the design of reinforced-concrete columns, where the effect of shrinkage stresses has been given consideration in the design formulas and allowable working stresses.

G-2: *Notation.*—The symbols and notation used in these regulations are defined as follows:

a = width of face of column or pedestal;

α = angle between inclined web bars and axis of beam;

A = total net area of column, footing, or pedestal, exclusive of fireproofing;

A' = loaded area of pedestal, pier or footing;

A_c = $A\ (1-p)$ = net area of concrete core of column (core area minus reinforcement);

A'_c = net area of concrete in columns with lateral ties (total column area minus area of reinforcement);

A_s = effective cross-sectional area of metal reinforcement in tension in beams or compression in columns; and the effective cross-sectional area of metal reinforcement which crosses any of the principal design sections of a flat slab and which meets the requirements of Sec. K-12;

A_v = total area of web reinforcement in tension within a distance of s (s_1, s_2, s_3, etc.) or the total area of all bars bent up in any one plane;

b = width of rectangular beam or width of flange of T-beam;

b_1 = dimension of the dropped panel of a flat slab in the direction parallel to l_1;

c = base diameter of the largest right circular cone which lies entirely within the column (including the capital) whose vertex angle is 90 deg. and whose base is 1½ in. below the bottom of the slab or the bottom of the dropped panel;

c = projection of footing from face of column;

d = depth from compression surface of beam or slab to center of longitudinal tension reinforcement;

E_c = modulus of elasticity of concrete in compression;

E_s = modulus of elasticity of steel in tension = 30,000,000 lb. per sq. in.;

f_c = compressive unit stress in extreme fiber of concrete;

f'_c = ultimate compressive strength of concrete at age of 28 days, based on tests of 6 by 12-in. or 8 by 16-in. cylinders made and tested in accordance with the Standard Methods of Making and Storing Specimens of Concrete in the Field and the Tentative Methods of Making Compression Tests of Concrete, of the A. S. T. M.

f_r = compressive unit stress in metal core;

f_s = tensile unit stress in longitudinal reinforcement;

f_v = tensile unit stress in web reinforcement;

F = total tension in a bar;

F' = total tensile stress in a bar developed in the length "y." See Sec. J-3.

h = unsupported length of column;

I = moment of inertia of a section about the neutral axis for bending;

l = span length of beam or slab (generally distance from center to center of supports; for special cases, see Sec. H-3 and K-6);

l = span length of flat slab, center to center of columns, in the rectangular direction in which moments are considered (see Sec. K-3);

l_1 = span length of flat slab, center to center of columns, perpendicular to the rectangular direction in which moments are considered;

M = bending moment or moment of resistance in general;

M_o = sum of positive and negative bending moments in either rectangular direction, at the principal design sections of a panel of a flat slab;

n = E_s/E_c = ratio of modulus of elasticity of steel to that of concrete;

Σo = sum of perimeters of bars in one set;

p = ratio of effective area of tension reinforcement to effective area of concrete in beams = A_s/bd; and the ratio of effective area of longitudinal reinforcement to the area of the concrete core in columns;

P = total safe axial load on column whose h/R is less than 40;

P' = total safe axial load on long column;

r_a = permissible working stress in concrete over the loaded area of a pedestal, pier or footing;

R = ratio of positive or negative moment in two column strips or one middle strip of a flat slab, to M_o;

R = least radius of gyration of a section;

s = spacing of web members, measured at the mid-depth of the beam and in the direction of the longitudinal axis of the beam;

t = thickness of flange of T-beam;

t_1 = thickness of flat slab without dropped panels or thickness of a dropped panel;

t_2 = thickness of flat slab with dropped panels at points away from the dropped panel;

u = bond stress per unit of area of surface of bar;

v = shearing unit stress;

V = total shear;

w = uniformly distributed load per unit of length of beam or slab;

w = upward reaction per unit of area of base of footing;

w' = uniformly distributed dead- and live-load per unit of area of a floor or roof;

W = total dead- and live-load uniformly distributed over a single panel area;

x = length of bar added for anchorage, including the hook, if any;

y = distance from the point at which the tension is computed to the point of beginning of anchorage.

G-3: *Unit Stresses in Terms of Ultimate Strength of Concrete.*—As specified in Sec. D-1, the structural drawings and plans shall show the ultimate strength of concrete for which the several portions of the structures were designed. The working stresses to be used in the computations shall be based on the ultimate strength indicated on the drawings and in the ratios shown in the succeeding sections of these regulations. The ultimate strength is designated as f'_c and refers to the ultimate strength at 28 days, based on 6 by 12-in. or 8 by 16-in. cylinders made, cured and tested in accordance with the Standard Methods of Making and Storing Specimens of Concrete in the Field and Tentative Methods of Making Compression Tests, of the American Society for Testing Materials in effect at the time of adoption of these regulations. These working stresses in the concrete are here summarized together with the permissible working stresses in the reinforcement.

SUMMARY OF WORKING STRESSES.

Direct Stresses in Concrete.

In columns; varies, see Sections L-3 and L-4.
In long columns; see Section L-8.
In piers and pedestals ..... $0.25f'_c$

Flexural Stresses in Concrete.

Extreme fibre stress in compression in flexure ..... $0.40f'_c$
Extreme fibre stress in compression in flexure adjacent to supports of continuous beams ..... $0.45f'_c$

Shearing Stresses in Concrete.

Beams without special anchorage of longitudinal reinforcement:
  With no web reinforcement ..... $0.02f'_c$
  With stirrups or bent-up bars or combinations of the two ..... $0.02f'_c$ to $0.06f'_c$
Beams with special anchorage of longitudinal reinforcement.
  With no web reinforcement ..... $0.03f'$
  With stirrups or bent-up bars or combinations of the two ..... $0.03f'_c$ to $0.12f'_c$
In flat slabs:
  At distance d from edge of column cap or dropped panel, see Sec. I-6 ..... $0.025f'_c$ to $0.03f'_c$

In footings:

Where longitudinal bars have no special anchorage	$0.02f'_c$
Where longitudinal bars have special anchorage	$0.03f'_c$

Bond Between Concrete and Reinforcement.

In beams and slabs, plain bars	$0.04f'_c$
In beams and slabs, deformed bars	$0.05f'_c$
In footings, plain bars, one way	$0.04f'_c$
In footings, plain bars, two ways	$0.03f'_c$
In footings, deformed bars, one way	$0.05f'_c$
In footings, deformed bars, two ways	$0.0375f'_c$

Stresses in Reinforcement.

Tension

(a) Billet-steel bars:	
(1) Structural steel grade	16,000 lb. per sq. in.
(2) Intermediate grade	18,000 " " " "
(3) Hard grade (where permitted)	18,000 " " " "
(b) Rail-steel bars (where permitted)	18,000 " " " "
(c) Structural steel shapes	16,000 " " " "
(d) Other steel reinforcement, 45 per cent of the yield point stress but not to exceed	18,000 " " " "

Compression

(a) Bars	nf_c
(b) Structural steel core of composite column	16,000 lb. per sq. in.
reduced for slenderness ratio	(See Section L-6)
(c) Structural steel column	16,000 lb. per sq. in.
reduced for slenderness ratio	(See Section L-7)
Composite cast-iron column	10,000 lb. per sq. in.
reduced for slenderness ratio	(See Section L-6)

CHAPTER H.

FLEXURAL COMPUTATIONS AND MOMENT COEFFICIENTS.

H-1: *Formulas for Flexure.*—Computations of flexural resistance of reinforced-concrete beams and slabs shall be based on the assumptions of Sec. G-1. The customary formulas or their equivalent shall be used.

H-2: *Working Stresses in Flexure.*—See Sec. G-3.

H-3: *Span Length.*—The span length of freely supported beams and slabs shall be the clear span plus the depth of beam or slab but shall not exceed the distance between centers of the supports.

The span length for continuous or restrained beams built to act integrally with supports shall be the clear distance between faces of supports.

For continuous or restrained beams having brackets built to act integrally with both beam and support and of a width not less than the width of the beam and making an angle of 45 deg. or more with the horizontal the span shall be measured from the section where the combined

depth of the beam and bracket is at least one-third more than the depth of the beam. No portion of such a bracket shall be considered as adding to the effective depth of the beam.

Maximum negative moments are to be considered as existing at the ends of the span, as defined above.

H-4: *Unsupported Length of Beams.*—The distance between lateral supports of the compression area of a beam shall not exceed 24 times the least width of compression flange.

H-5: *Requirements for T-Beams.*—In T-beam construction the slab shall be built integral with the beam. The effective flange width to be used in the design of symmetrical T-beams shall not exceed one-fourth of the span length of the beam, and its overhanging width on either side of the web shall not exceed eight times the thickness of the slab nor one-half the clear distance to the next beam.

For beams having a flange on one side only, the effective flange width to be used in design shall not exceed one-tenth of the span length of the beam, and its overhanging width from the face of the web shall not exceed six times the thickness of the slab, nor one-half the clear distance to the next beam.

Where the principal slab reinforcement is parallel to the beam, transverse reinforcement, not less in amount than 0.3 per cent of the sectional area of the slab, shall be provided in the top of the slab and shall extend across the beam and into the slab not less than one-fourth of the clear span of the slab measured parallel to the beam. The spacing of the bars shall not exceed 18 in.

Provision shall be made for the compressive stress at the support in continuous T-beam construction.

The overhanging portion of the flange of the beam shall not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

Isolated beams in which the T-form is used only for purpose of providing additional compression area, shall have a flange thickness not less than one-half the width of the web and a total flange width not more than four times the web thickness.

H-6: *Moment Coefficients for the Usual Conditions.*—(A) **Freely supported or slightly restrained continuous beams or slabs of equal span; uniform load.** Beams and slabs of equal spans freely supported or built to act integrally with beams, girders or other slightly restraining support, or beams and slabs built into brick or masonry walls in a manner which develops only partial end restraint, and carrying uniformly distributed loads shall be designed for the following moments at critical sections:

(a) Beams and slabs of one span,
Maximum positive moment near center,

$$M = \frac{wl^2}{8} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

(*b*) Beams and slabs continuous for two spans only,

(1) Maximum positive moment near center,

$$M = \frac{wl^2}{10} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

(2) Negative moment over interior support,

$$M = \frac{wl^2}{8} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

(*c*) Beams and slabs continuous for more than two spans,

(1) Maximum positive moment near center and negative moment at support of interior spans,

$$M = \frac{wl^2}{12} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

(2) Maximum positive moment near centers of end spans and negative moment at first interior support,

$$M = \frac{wl^2}{10} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

(*d*) Negative moment at end supports for cases (*a*), (*b*), (*c*) of this section,

$$M = \text{not less than } \frac{wl^2}{16} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

(B) Fully restrained continuous beams or slabs of equal span: uniform load.

Beams and slabs of equal span built to act integrally with columns, walls, or other restraining supports and assumed to carry uniformly distributed loads, shall (except as provided in (A)) be designed for the following moments at critical sections:

(*a*) Interior spans,

(1) Negative moment at interior supports except the first,

$$M = \frac{wl^2}{12} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

(2) Maximum positive moment near centers of interior spans,

$$M = \frac{wl^2}{16} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

(*b*) End spans of continuous beams and slabs and beams and slabs of which I/l is less than twice the sum of the values of I/h for the exterior columns above and below which are built into the beams:

(1) Maximum positive moment near center of span and negative moment at first interior supports,

$$M = \frac{wl^2}{12} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

(2) Negative moment at exterior supports,

$$M = \frac{wl^2}{12} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

(c) End spans of continuous beams, and beams of one span, in which I/l is equal to or greater than twice the sum of the values of I/h for the exterior column above and below which are built into the beams:

(1) Maximum positive moment near center of span and negative moment at first interior support,

$$M = \frac{wl^2}{10} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

(2) Negative moment at exterior support,

$$M = \frac{wl^2}{16} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

In (B) and (C) "I" represents the moment of inertia which, for those calculations, shall be computed on the assumption that the member is homogeneous, neglecting the reinforcement but including that portion of the concrete section outside of the reinforcement which is ordinarily considered as fireproofing. l and h are the span length and column height respectively as defined in Section H-3 and L-2.

(C) Continuous beams or slabs of unequal span or with non-uniform loads.

Continuous beams with unequal spans, or with other than uniformly distributed loading, whether freely-supported or restrained, shall be designed for the actual moments under the conditions of loading and restraint.

Provision shall be made where necessary for negative moment near the center of short spans which are adjacent to long spans, and for the negative moment at the end supports, if restrained.

CHAPTER I.

SHEAR AND DIAGONAL TENSION.

I-1: *Shearing Unit Stress.*—Shearing stresses themselves are seldom important in the design of beams or slabs. However, since they are a convenient measure of diagonal tension which must be provided for, the provisions of this chapter are expressed in terms of shear. The shearing unit stress, v, in reinforced-concrete beams shall be computed by Formula 14:

$$v = \frac{8\,V}{7\,bd} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

The shearing unit stress shall be computed on the minimum width of rectangular beams and on the minimum thickness of the web in beams of I- or T-section.

In tile and joist construction, the shearing unit stress shall be computed on a width equal to the thickness of the concrete web plus the thickness of the vertical webs of the concrete or clay tile in contact with the joist.

I-2: *Types of Web Reinforcement.*—Web reinforcement may consist of:

(a) Vertical stirrups or web reinforcing bars;

(b) Inclined stirrups or web reinforcing bars forming an angle of 30 deg. or more with the axis of the beam.

(c) Longitudinal bars bent up at an angle of 15 deg. or more with the axis of the beam.

Stirrups or bent up bars to be considered effective as web reinforcement shall be anchored at both ends, according to the provisions of Sec. J-5.

I-3: *Spacing of Stirrups or Bent-Up Bars.*—Where the shearing stress is not greater than $0.06f'_c$ the distance s measured in the direction of the axis of the beam between two successive stirrups or bent-up bars, shall not exceed the value given by Formula 15,

$$s = \frac{45\,d}{a} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

where a is the angle in degrees between the inclined web bars and the axis of the beam.

Bent-up bars and stirrups shall be considered effective in reinforcing the web only within the area between two vertical planes distant $s/2$ in either direction from the point where the bent-up bar crosses the mid-depth of the beam.

Where the shearing stress is greater than $0.06f'_c$ the distance s shall not be greater than two-thirds of the value given by Formula 15.

I-4: *Maximum Shearing Unit Stress in Beams.*—In beams in which the longitudinal reinforcement is without special anchorage, the shearing unit stress computed by Formula 14 shall not exceed the value given by Formula 16 and in no case shall it exceed $0.06f'_c$.

$$v = 0.02f'_c + \frac{f_v A_v}{bs}\left(\sin a + \cos a\right) \qquad \ldots\ldots\ldots\ldots (16)$$

In beams in which the longitudinal reinforcement is anchored as specified in Sec. J-5, the shearing unit stress shall not exceed the value given by Formula 16 when $0.03f'_c$ is substituted for $0.02f'$, and in no case shall it exceed $0.12f'_c$.

Where the entire web reinforcement consists of longitudinal bars bent up in a single plane the allowance for the quantity $\frac{f_v A_v}{bs}$ (sin a + cos a)

in Formula 16 shall not exceed 75. Such bent-up reinforcement shall cross the mid-depth of the beam at a distance not greater than $s/2$ from the face of the support and shall not be considered effective in resisting diagonal tension over a distance greater than s from the support, where s has the value by Formula 15 of Sec. I-3.

I-5: *Combined Web Reinforcement.*—Where two or more types of web reinforcement are used in conjunction, the total shearing resistance of the beam shall be assumed as the sum of the shearing resistances computed for the various types separately. In such computations the shearing resistance of the concrete (the term $0.02f'_c$ or $0.03f'_c$ in Formula 16) shall be included only once. In no case shall the maximum shearing stress be greater than the limiting values in Sec. I-4.

I-6: *Shearing Stress in Flat Slabs.*—In flat slabs, the shearing unit stress computed by Formula 14 (in which d shall be taken as $t_1 - 1\frac{1}{2}$) on a vertical section which lies at a distance $t_1 - 1\frac{1}{2}$ from the edge of the column capital and parallel with it shall not exceed $0.02f'_c$ multiplied by the following factor: 1 plus the ratio which the cross-sectional area of the negative reinforcement in the width of strip directly above the column capital bears to the cross-sectional area of the negative reinforcement in the full width of two column strips. At least 25 per cent of the total cross-sectional area of the negative reinforcement in two column strips must be within the width of strip directly above the column capital.

In no case shall the unit shearing stress exceed $0.03f'_c$.

The shearing unit stress computed by Formula 14 (in which d shall be taken as $t_2 - 1\frac{1}{2}$) on a vertical section which lies at a distance of $t_2 - 1\frac{1}{2}$ from the edge of the dropped panel and parallel with it shall not exceed $0.03f'_c$. At least 50 per cent of the cross-sectional area of the negative reinforcement in two column strips must be within the width of strip directly above the dropped panel.

I-7: *Shear and Diagonal Tension in Footings.*—The shearing stress shall be taken as not less than that computed by Formula 14. The stress on the critical section shall not exceed $0.02f'_c$ for footings with straight bars, nor $0.03f'_c$ for footings in which the bars are anchored at both ends by adequate hooks or otherwise as specified in Sec. J-5.

The critical section for diagonal tension in footings on soil shall be computed on a vertical section through the perimeter of the lower base of a frustum of a cone or pyramid which has a base angle of 45 deg., and which has for its top the base of the column or pedestal and for its lower base the plane at the centroid of longitudinal reinforcement.

The critical section for diagonal tension in footings on piles shall be computed on a vertical section at the inner edge of the first row of piles entirely outside a section midway between the face of the column or pedestal and the section defined for soil footings, but in no case outside of that section. For piles not arranged in rows, the critical section shall be taken midway between the face of the column and the section defined for soil footings.

CHAPTER J

BOND AND ANCHORAGE.

J-1: *Computation of Bond Stress in Beams.*—Where bar reinforcement is used to resist tensile stresses developed by beam action, the bond stress shall be taken as not less than that computed by Formula 17.

$$u = \frac{8\,V}{7\,\Sigma o d} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

For continuous or restrained members, the critical section for bond for the positive reinforcement shall be assumed to be at the point of inflection, that for the negative reinforcement shall be assumed to be at the face of the support, and at the point of inflection. For simple beams or at the outer ends of freely supported end spans of continuous beams, the critical section for bond shall be assumed to be at the face of the support.

Bent-up longitudinal bars which, at the critical section, are within a distance $\frac{d}{3}$ from horizontal reinforcement under consideration may be included with the straight bars in computing Σo.

In footings only the bars specified in Sec. M-4 as effective in resisting bending moment shall be considered as resisting bond stresses. Special investigation shall be made of bond stresses in footings with stepped or sloping upper surface, as maximum bond stresses may occur at the vertical plane of the steps or near the edges of the footing.

J-2: *Permissible Bond Stress: Ordinary Anchorage.*—In beams where the ordinary anchorage described in Sec. J-4 is provided, the bond stress computed by Formula 17 at any section shall not exceed the following values:

For plain bars ..... $u = 0.04f'_c$

For deformed bars meeting the requirements of Section C-7 ..... $u = 0.05f'_c$

The permissible bond stress for footings and similar members in which reinforcement is placed in more than one direction shall not exceed 75 per cent of the above values. Small percentages of reinforcement added for temperature or shrinkage stresses shall not be interpreted as requiring this reduction.

J-3: *Permissible Bond Stress: Special Anchorage.*—In members in bending, bond stresses (computed by Formula 17) exceeding those specified in Sec. J-2, but in no case more than 2½ times the latter, may be used, provided that sufficient additional length of bar is added beyond the theoretical point of zero moment (end of span or point of inflection) to provide for the development of the excess in bond stress over that specified in Sec. J-2. The length x to be added for this purpose may be expressed algebraically by Formula 18.

$$x u \Sigma o = F - F' \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

where, x = the length of bar added for anchorage, including the hook, if any,
u = permissible bond stress specified in Sec. J-2,
Σo = the perimeter of the bar or bars under consideration,
F = total tension in the bar or bars under consideration,
F' = the total tension in the bar which would be developed in the length y by the computed bond stresses, except that no values greater than those specified in Sec. J-2 be used in the computation, F' = bond stress times Σoy.
y = the distance from the point at which the tension is computed to the point of beginning of anchorage.

The point of beginning of anchorage shall be taken at the edge of support for freely supported beams, and at the point of inflection (for the loading condition under consideration) for fixed or continuous beams; anchorage of negative reinforcing to be toward the center of the beams from this point.

The length of bar added for anchorage may be either straight or bent. The radius of bend shall not be less than four bar diameters.

J-4: *Ordinary Anchorage Requirements.*—In continuous, restrained or cantilever beams, the length of anchorage x of the tensile negative reinforcement beyond the face of the support shall provide for the full maximum tension by Formula 18 in which u equals the value given in Sec. J-2, and, for this case, since $y = o$, $F' = o$. Such anchorage shall provide a length of bar not less than the depth of the beam. In the case of end supports which have a width less than three-fourths of the depth of the beam, the bars shall be bent down toward the support a distance not less than the effective depth of the beam. The portion of the bar so bent down shall be as near to the end of the beam as protective covering permits. In continuous or restrained beams, negative reinforcement shall be carried to or beyond the point of inflection. Not less than one-fourth of the area of the positive reinforcement shall extend into the support to provide an embedment of ten or more bar diameters.

In simple beams or at the outer ends of freely supported end spans of continuous beams at least one-fourth of the area of the tensile reinforcement shall extend along the tension side of the beam and beyond the face of the support to provide an embedment of ten or more bar diameters.

J-5: *Special Anchorage Requirements.*—Where increased shearing stresses are used as provided in Secs. I-3 and I-6 or increased bond stresses as provided in Sec. J-3, special anchorage of all reinforcement in addition to that required in Sec. J-4 shall be provided as follows:

(a) In continuous and restrained beams, anchorage beyond points of inflection of at least one-third the area of the negative reinforcement and beyond the face of the support of at least one-third the area of the positive reinforcement, shall be provided to develop one-third of the maximum working stress in tension. The anchorage length x shall be computed by

Formula 18 ($y = o$, therefore $F' = 0$) with bond stresses not greater than those specified in Sec. J-2.

(b) At the edges of footings, anchorage for all the bars for one-third the maximum working stress in tension shall be provided within a region where the tension in the concrete, computed as an unreinforced beam, does not exceed 40 lb. per sq. in. In any case the reinforcement bars shall extend to within 4 in. of the edge of the footing but not closer than 3 in. as specified in Sec. F-6.

(c) In simple beams or at the outer ends of freely supported end spans of continuous beams, at least one-half of the tensile reinforcement shall extend along the tension side of the beam to provide an anchorage beyond the face of the support for one-third of the maximum working stress in tension.

J-6: *Anchorage of Web Reinforcement.*—Web bars shall be anchored at both ends by:

(a) providing continuity with the longitudinal reinforcement; or
(b) bending around the longitudinal bar; or
(c) a semi-circular hook which has a radius not less than four times the diameter of the web bar.

Stirrup anchorage shall be so provided in the compression and tension regions of a beam as to permit the development of safe working tensile stress in the stirrup at a point $0.3d$ from either face. (Generally a properly-anchored stirrup whose diameter does not exceed one-fiftieth of the depth of the beam will meet these requirements.)

The end anchorage of a web member not in bearing on the longitudinal reinforcement shall be such as to engage an amount of concrete sufficient to prevent the bar from pulling out. In all cases the stirrups shall be carried as close to the upper and lower surfaces as fireproofing requirements permit.

CHAPTER K

FLAT SLABS.

(Two-Way and Four-Way Systems with Square or Rectangular Panels.)

K-1: *Limitations.*—The term flat slabs as used in these regulations, refers to concrete slabs, having reinforcement bars extending in two or four directions, without beams or girders to carry the load to supporting members. The moment coefficients, moment distribution, and slab thicknesses specified herein are for slabs which have three or more rows of panels in each direction, and in which the panels are approximately uniform in size. Slabs with paneled ceiling or with depressed paneling in the floor shall be considered as coming under the requirements herein given, provided the depth of the thicker portions of the slab does not exceed 1.5 times the depth of the remainder of the slab.

These regulations shall not apply to flat slabs in which the ratio of length to width of panel exceeds 1.4.

K-2: *Panel Strips and Principal Design Section.*—For convenience of reference, a flat slab panel shall be considered as consisting of strips as follows:

A *middle strip* one-half panel in width, symmetrical with respect to the panel center line and extending through the panel in the direction in which moments are being considered;

Two *column strips*, each one-quarter panel in width, occupying the two quarter panel areas outside of the middle strip.

When considering moments in the direction of the width of the panel, the panel is similarly divided by strips, the widths of which are respectively one-half and one-quarter of the length of the panel.

In the succeeding paragraphs, the provisions for limiting moments, etc., are related to certain critical sections. These sections are referred to as the principal design sections and are located as follows:

Sections for Negative Moment. These shall be taken along the edges of the panel, that is, along the lines joining the column centers. For the column strips, the section shall follow the center line between columns to the edge of the column capital (i. e., to a point $c/2$ from the column center) and then around the circumference of the column capital for a one-quarter circumference.

Sections for Positive Moment. These shall be taken on the center-line of the panel, crossing the strips for which moments are being considered.

K-3: *Moments in Interior Panels.*—In flat slabs in which the ratio of reinforcement (p) for negative moment in the column strip is not greater than 0.01, the numerical sum of the positive and negative moments in the direction of either side of a rectangular panel shall be not less than that given by Formula 19.

$$M_o = 0.09\ Wl\left(1 - \frac{2c}{3l}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

where M_o = sum of positive and negative bending moments, at the principal design sections, in the direction in which the length is given by l. This moment is in foot-pounds where the other items are in the units indicated below;

c = base diameter in feet of the largest right circular cone which lies entirely within the column (including the capital) the vertex angle of which is 90 deg. and the base of which is 1½ in. below the bottom of the slab or the bottom of the dropped panel;

l = span length in feet of the flat slab panel, center to center of columns in the direction in which moments are considered. (When considering moments in any direction, the width of column and middle strips must be related to the length of span at right angles to that in which moments are being considered.)

W = total dead- and live-load in pounds uniformly distributed over a single panel area.

K-4: *Moments in Principal Design Sections.*—The moments in the principal design sections shall be those given in the accompanying table of moments, except as follows:

(a) The sum of the maximum negative moments in the two column strips may be greater or less than the values given in table of moments by not more than $0.03\,M_o$.

(b) The maximum negative moment and the maximum positive moments in the middle strip and the sum of the maximum positive moments in the two-column strips may be greater or less than the values given in table of moments by not more than $0.01\,M_o$.

MOMENTS TO BE USED IN DESIGN OF FLAT SLABS.
For Interior Panels Fully Continuous.

Strip	Flat Slabs without Dropped Panels		Flat Slabs with Dropped Panels	
	Negative	Positive	Negative	Positive
Slabs with 2-way Reinforcement.				
Column strip	$0.23\,M_o$	$0.11\,M_o$	$0.25\,M_o$	$0.10\,M_o$
2-Column strips	$0.46\,M_o$	$0.22\,M_o$	$0.50\,M_o$	$0.20\,M_o$
Middle strip	$0.16\,M_o$	$0.16\,M_o$	$0.15\,M_o$	$0.15\,M_o$
Slabs with 4-way Reinforcement.				
Column strip	$0.25\,M_o$	$0.10\,M_o$	$0.27\,M_o$	$0.095\,M_o$
2-Column strips	$0.50\,M_o$	$0.20\,M_o$	$0.54\,M_o$	$0.190\,M_o$
Middle strip	$0.10\,M_o$	$0.20\,M_o$	$0.08\,M_o$	$0.190\,M_o$

K-5: *Lateral Dimensions of Dropped Panels.*—The dropped panel shall have a length or diameter in each direction parallel to a side of the panel of not less than one-third the panel length in that direction.

K-6: *Thickness of Slabs and Dropped Panels.*—The total thickness of the slab through the dropped panel, t_1, in inches, or of the slab if a dropped panel is not used, shall be not less than the value given by Formula 20.

$$t_1 = 0.038\left(1 - 1.44\frac{c}{l}\right) l \sqrt{Rw'\frac{l_1}{b_1}} + 1\tfrac{1}{2} \quad \ldots\ldots\ldots\ldots (20)$$

where R = ratio of negative moment in the two-column strips parallel to the length to the total moment M_o;

w' = uniformly distributed dead- and live-load per sq ft.;

l_1 = width in feet of the panel at right angles to the direction of the length l;

b_1 = dimension in feet of the dropped panel in the direction parallel to l_1, except that in a slab without dropped panel b_1 shall be taken as $0.5\,l_1$.

For slabs with dropped panels the total thickness in inches at points beyond the dropped panel shall be not less than

$$t_2 = 0.02\, l \sqrt{w'} + 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

The dropped panel shall have a thickness, t_1, not greater than 1.5 t_2.

In determining minimum thickness by Formulas 20 and 21, the value of l shall be the panel length center to center of the columns, on the long side of panel, and the value of l_1 shall be the panel width, center to center of the columns.

The slab thickness t_1 or t_2 shall in no case be less than $l/32$ for floor slabs, and not less than $l/40$ for roof slabs.

K-7: *Wall and Other Irregular Panels.*—In wall panels and other panels in which the slab is not continuous with an adjacent panel, the maximum negative moment at the edge of the panel opposite to the discontinuous edge and the maximum positive moment at the center of this panel shall be increased as follows:

(a) In the column strip perpendicular to the wall or discontinuous edge, 15 per cent greater than that given in the table of moments for interior panels;

(b) Middle strip perpendicular to wall or discontinuous edge, 30 per cent greater than that given in the table of moments for interior panels.

In these strips the bars used for positive moments perpendicular to the discontinuous edge shall extend to the edge of the panel at which the slab is discontinuous.

At the wall or discontinuous edge the negative moment in the column strip shall be taken as not less than 90 per cent and in the middle strip not less than 65 per cent of the corresponding moments for a normal interior panel as given in the table of Sec. K-4.

K-8: *Panels with Marginal Beams.*—In panels having a marginal beam on one edge or on each of two adjacent edges, the beam shall be designed to carry at least the load superimposed directly upon it, exclusive of the panel load. A marginal beam which has a depth greater than the thickness of the dropped panel into which it frames, shall be designed to carry, in addition to the load superimposed directly upon it, a uniformly distributed load equal to at least one-fourth of the total live- and dead-load for which the adjacent panel or panels are designed. Slabs supported by marginal beams on opposite edges shall be designed as freely supported slabs for the entire load.

Column strips adjacent to and parallel with marginal beams having a depth less than the thickness of the dropped panel, shall be designed to resist the moment specified for a column strip in the table of moments. Column strips adjacent to and parallel with marginal beams having a depth greater than the thickness of the dropped panel, shall be designed

to resist a moment at least one-half as great as that specified for a column strip in the table of moments.

In wall columns where brackets are used in place of capitals, the value of c in the direction in which the bracket extends, shall be taken as twice the distance from the center of the column to a point 1½ in. back from the edge of the bracket and averaged with the value of c for an interior column capital in the computation for moment in Formula 19. The value of c for column strips parallel and adjacent to marginal beams shall be taken as equal to the width of the wall column if no bracket is used in this direction.

K-9: *Flat Slabs on Bearing Walls.*—Where there is a beam or a bearing wall at the center line of columns in the interior portion of a continuous flat slab, the negative moment at the beam or wall line in the middle strip perpendicular to the beam or wall shall be taken as 30 per cent greater than the negative moment specified in the table of moments (Sec. K-4) for a middle strip. The column strip adjacent to and lying on either side of the beam or wall shall be designed to resist moments at least one-half of those specified in the table of moments (Sec. K-4) for a column strip.

K-10: *Point of Inflection.*—In the middle strip the point of inflection for the slabs without dropped panels shall be assumed at a line 0.30 l distant from the center of the span and for slabs with dropped panels 0.25 l distant from the center of the span.

In the column strip the point of inflection for slabs without dropped panels shall be at a line 0.30 $(l - c)$ distant from the center of the panel and 0.25 $(l - c)$ for slabs with dropped panels.

K-11: *Effective Reinforcement.*—The reinforcement which crosses any section and which fulfills the requirements given in Sec. K-12 may be considered as effective in resisting the moment at the section. The sectional area of a bar multiplied by the cosine of the angle between the direction of the axis of the bar and any other direction may be considered effective as reinforcement in that direction.

K-12: *Arrangement of Reinforcement.*—Provision shall be made for securing the reinforcement in place so as to resist properly not only the critical moments, but also the moments at intermediate sections. Provision shall also be made for possible shifting of the point of inflection by carrying all bars in rectangular or diagonal directions, to points at least 20 diameters beyond the point of inflection each side of a section of critical moment, either positive or negative. Lapped splices shall not be permitted at or near regions of maximum stress except as described in Sec. F-5. At least four-tenths of all bars in each direction shall be of such length and shall be so placed as to provide reinforcement at two sections of critical negative moment and at the intermediate section of critical positive moment. Not less than one-third of the bars used for positive reinforcement in the column strip shall extend into the dropped panel at least 20 diameters of the bar, or in case no dropped panel is used, shall extend to within

one-eighth of the span length from the center line of the column or the support.

K-13: *Special Panel Arrangement.*—For structures having a width of less than three (3) rows of panels, or in which irregular or special panels are used, an analysis shall be made of the moments developed in both slabs and columns. When so required, computations shall be submitted to the commissioner of buildings for approval.

CHAPTER L.

REINFORCED CONCRETE COLUMNS.

L-1: *Limiting Dimensions.*—Unless designed as long columns under the provisions of Sec. L-8, reinforced concrete columns shall not be longer than 40 times the least radius of gyration ($40R$). Principal columns in buildings shall have a minimum diameter or thickness of 12-in. Posts that are not continuous from story to story shall have a minimum diameter or thickness of 6 in.

L-2: *Unsupported Length and Radius of Gyration of Columns.*—The unsupported length of reinforced concrete columns shall be taken as:

(*a*) In flat slab construction the clear distance between the floor and under side of the capital;

(*b*) In beam-and-slab construction, the clear distance between the floor and the under side of the shallowest beam framing into the column at the next higher floor level;

(*c*) In floor construction with beams in one direction only, the clear distance between floor slabs;

(*d*) In columns supported laterally by struts or beams only, the clear distance between consecutive pairs (or groups) of struts or beams, provided that to be considered an adequate support, two such struts or beams shall meet the column at approximately the same level and the angle between the two planes formed by the axis of the column and the axis of each strut respectively is not less than 75 deg. nor more than 105 deg.

When haunches are used at the junction of beams or struts with columns, the clear distance between supports may be considered as reduced by two-thirds of the depth of the haunch.

The radius of gyration of a column shall be computed from the concrete area of the core and the transformed section of the longitudinal steel area, that is, the actual area of steel multiplied by n, this assumed to be distributed uniformly around the periphery of the core.

L-3: *Design of Spiral Columns.*—The safe axial load on columns reinforced with longitudinal bars and closely spaced spirals enclosing a circular core, shall not be greater than that determined by Formula (22).

$$P = A\ [1 + (n-1)p]\ [300 + (0.10 + 4p)f'_c] \quad \ldots\ldots\ldots\ldots (22)$$

where P = total safe axial load on column in which h/R (the unsupported length divided by the radius of gyration) is less than 40;

A = area of the concrete core enclosed within the spiral; the diameter of the core (or of the spiral) shall be taken as the distance center to center of the spiral wire;

p = ratio of effective area of longitudinal reinforcement to area of the concrete core;

$f'c$ = ultimate strength of concrete at 28 days as defined in Sec. G-3.

The longitudinal reinforcement shall consist of at least six bars of minimum diameter of ½ in., and its effective cross-sectional area shall not be less than 1 per cent nor more than 6 per cent of that of the core.

The spiral reinforcement shall be not less than one-fourth the volume of the longitudinal reinforcement. It shall consist of evenly spaced continuous spirals held firmly in place and true to line by at least three vertical spacer bars. The spacing of the spirals shall be not greater than one-sixth of the diameter of the core and in no case more than 3 in.

Reinforcement shall be protected everywhere by a covering of concrete cast monolithic with the core, which shall have a minimum thickness of 1½ in.

L-4: *Design of Columns with Lateral Ties.*—The safe axial load on columns reinforced with longitudinal bars and separate lateral ties shall be not greater than that determined by Formula (23).

$$P = (A_c' + A_s n) f_c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

where A'_c = net area of concrete in the column (total column area minus area of reinforcement);

A_s = effective cross-sectional area of longitudinal reinforcement; and

f_c = permissible compressive stress in concrete and shall not exceed $0.20f'_c$.

The amount of longitudinal reinforcement shall not be less than 0.5 per cent nor shall the amount considered in the calculations be more than 2 per cent of the total area of the column. The longitudinal reinforcement shall consist of not less than four bars of minimum diameter of ½ in., placed with clear distance from the face of the column not less than 2 in.

Lateral ties shall be at least ¼ in. in diameter spaced not more than 8 in. apart.

L-5: *Bending in Columns.*—The bending moments in interior and exterior columns shall be determined on the basis of loading conditions and end restraint, and shall be provided for in the design.

In flat slab construction, a bending moment at all interior columns equal to $W_1 l/40$ shall be assumed to cover all ordinary cases of unequal

loading in floors and roofs; where $W_1 =$ the total live-load on one panel and $l =$ the span center to center of columns lengthwise of the panels. For known eccentric loads or uneven spacing of columns, computation of moments shall be made accordingly. Resistance to these bending moments shall be provided in the columns immediately above and below in direct proportion to the values of their ratios of I/h (See Sec. H-6 and L-2). In columns supporting roofs the moment shall be resisted by the column below.

Wall columns in flat slab construction shall be designed to resist bending in the same manner as interior columns except that the total live- and dead-load in the panel shall be used instead of the live-load only when computing the moment. Any counter moment due to the weight of the structure that projects beyond the column center line may be deducted from the moment computed as just described.

The recognized methods shall be followed in calculating the stresses due to combined axial load and bending. The limiting unit stresses shall be as follows:

(a) **With Spiral Reinforcement**—The compressive unit stress at the extreme fibre on the concrete within the core area under combined axial load and bending shall not exceed the value given by the expression $300 + (0.10 + 4p)f'_c$.

(b) **With Lateral Ties**—Additional longitudinal reinforcement may be used if required to provide for the bending stresses, and the compressive unit stress at the extreme fibre on the concrete under combined axial load and bending may be increased to $0.30f'_c$. The column section, however, shall not be less than that required by the provisions of Sec. L-4 where axial load alone is considered. The total amount of reinforcement considered in the computations shall be not more than 4 per cent of the total area of the column.

Tension in the longitudinal reinforcement due to bending of the column shall not exceed 16,000 lb. per sq. in.

L-6: *Composite Columns.*—The safe load on composite columns in which a structural steel or cast-iron column is thoroughly encased in a circumferentially reinforced concrete core shall be based on a certain unit stress for the steel or cast-iron core plus a unit stress of $0.25f'_c$ on the area within the spiral core.

The unit compressive stress on the steel section shall be not greater than that determined by Formula 24.

$$f_r = 18{,}000 - 70\ h/R \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

but shall not exceed 16,000 lb. per sq. in.

The unit stress on the cast-iron section shall be not greater than that determined by Formula 25.

$$f_r = 12{,}000 - 60\ l/R \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

but shall not exceed 10,000 lb. per sq. in.

In Formulas 24 and 25,

f_r = compressive unit stress in metal core, and

R = least radius of gyration of the steel or cast-iron section.

The diameter of the cast-iron section shall not exceed one-half of the diameter of the core within the spiral. The spiral reinforcement shall be not less than 0.5 per cent of the volume of the core within the spiral and shall conform in quality, spacing and other requirements to the provisions for spirals in Sec. L-3.

Ample section of concrete and continuity of reinforcement shall be provided at the junction with beams or girders. The area of the concrete between the spiral and the metal core shall be not less than that required to carry the total floor load of the story above on the basis of a stress in the concrete of $0.35f'_c$, unless special brackets are arranged on the metal core to receive directly the beam or slab load.

L-7: *Structural Steel Columns.*—The safe load on a structural steel column of a section which fully encases an area of concrete, and which is protected by an outside shell of concrete at least 3 in. thick, shall be computed in the same manner as for composite columns in Sec. L-6, allowing $0.25f'_c$ on the area of the concrete enclosed by the steel section. The outside shell shall be reinforced by wire mesh weighing not less than 0.2 lb. per sq. ft. or by ties or spirals of equal weight, and with a maximum spacing of 6 in. between strands or hoops. Special brackets shall be used to receive the entire floor load at each story. The safe load in steel columns calculated by Formula 24 shall not exceed 16,000 lb. per sq. in.

L-8: *Long Columns.*—The permissible working load on the core in axially loaded columns which have a length greater than 40 times the least radius of gyration of the column core ($40R$) shall be not greater than that determined by Formula 26.

$$\frac{P'}{P} = 1.33 - \frac{l}{120\ R} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

where P' = total safe axial load on long columns;

P = total safe axial load on column of the same section whose l/R is less than 40, determined as in Sec. L-3 and L-4; and

R = least radius of gyration of column core as defined in Sec. L-2.

CHAPTER M.

FOOTINGS.

M-1: *General.*—The requirements for flexure, shear and bond of Chapters H, I and J shall govern the design of footings except as hereinafter provided.

M-2: *Loads.*—Footings resting directly on soil or on piles shall be proportioned as to area or number of piles on the basis of the total column load plus the weight of the footing itself. For computations of moments and shears, an upward reaction per unit area or per pile shall be based on the total column load (not including the weight of the footing itself) divided by the area or by the number of piles.

M-3: *Sloped or Stepped Footings.*—Footings in which the thickness has been determined by the requirements for shear as specified in Sec. I-7, may be sloped or stepped between the critical section and the edge of the footing, provided that the shear on no section outside the critical section exceeds the value specified, and provided further that the thickness of the footing above the reinforcement at the edge shall not be less than 6 in. for footings on soil nor less than 12 in. for footings on piles. Sloped or stepped footings shall be cast as a unit.

M-4: *Bending in Footings.*—The critical section for bending in a concrete footing which supports a concrete column or pedestal, shall be considered to be at the face of the column or pedestal. Where steel or cast-iron column bases are used, the moment in the footing shall be computed at the middle and at the edge of the base; the load shall be considered as uniformly distributed over the column or pedestal base.

The bending moment at the critical section in a square footing supporting a concentric square column, shall be computed from the load on the trapezoid bounded by one face of the column, the corresponding outside edge of the footing, and the portions of the two diagonals. The load on the two corner triangles of this trapezoid shall be considered as applied at a distance from the face equal to six-tenths of the projection of the footing from the face of the column. The load on the rectangular portion of the trapezoid shall be considered as applied at its center of gravity. The bending moment is expressed by Formula 27.

$$M = \frac{w}{2}(a + 1.2c)c^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

where M = bending moment at critical section of footing;
a = width of face of column or pedestal;
c = projection of footing from face of column; and
w = upward reaction per unit of area of base of footing.

For a round or octagonal column, the distance a shall be taken as equal to the side of a square of an area equal to the area enclosed within the perimeter of the column.

The reinforcement in each direction in the footing shall be determined as for a reinforced concrete beam with the limiting stresses specified in Sec. G-3; the effective depth shall be the distance from the top of the footing to the plane of the reinforcement. The sectional area of reinforcement shall be distributed uniformly across the footing unless the width is greater than the side of the column or pedestal plus twice the effective depth of the footing, in which case the width over which the reinforcement is spread may be increased to include one-half the remaining width of the footing. In order that no considerable area of the footing shall remain unreinforced, additional reinforcement shall be placed outside of the width specified, but such reinforcement shall not be considered as effective in resisting the calculated bending moment. For the extra reinforcement a spacing double that within the effective belt may be used.

The extreme fiber stress in compression in the concrete shall be kept within the limits specified in Sec. G-3. The extreme fiber stress in sloped or stepped footings shall be based on the exact shape of the section for a width not greater than that assumed effective for reinforcement.

M-5: *Footings Other than Square.*—A rectangular or irregularly shaped footing shall be computed by dividing it into rectangles or trapezoids tributary to the sides of the column, using the distance to the center of gravity of the area as the moment arm of the upward forces. Outstanding portions of combined footings shall be treated in the same manner. Other portions of combined footings shall be designed as beams or slabs.

M-6: *Shearing and Bond Stresses.*—See Sec. I-7 and J-1 to 5 inclusive.

M-7: *Transfer of Stress at Base of Column.*—The compressive stress in longitudinal reinforcement at the base of a column shall be transferred to the pedestal or footing by either dowels or distributing bases. When dowels are used, there shall be at least one for each column bar, and the total sectional area of the dowels shall be not less than the sectional area of the longitudinal reinforcement in the column. The dowels shall extend into the column and into the pedestal or footing not less than 50 diameters of the dowel bars for plain bars, or 40 diameters for deformed bars.

When metal distributing bases are used, they shall have sufficient area and thickness to transmit safely the load from the longitudinal reinforcement in compression and bending. The permissible compressive unit stress on top of the pedestal or footing directly under the column shall be not greater than that determined by Formula 28.

$$r_a = 0.25 f'_c \sqrt[3]{\frac{A}{A'}} \qquad (28)$$

where r_a = permissible working stress over the loaded area;

A = total area at the top of the pedestal or footing;

A' = loaded area at the column base;

f'_c = ultimate compressive strength of concrete. (See Sec. D-1).

In sloped or stepped footings A may be taken as the area of the top horizontal surface of the footing or as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base the loaded area A', and having side slopes of 1 vertical to 2 horizontal.

M-8: *Pedestals without Reinforcement.*—The allowable compressive unit stress on the gross area of a concentrically loaded pedestal or on the minimum area of a pedestal footing shall not exceed $0.25f'_c$, unless reinforcement is provided and the member designed as a reinforced-concrete column.

The depth of a pedestal or pedestal footing shall be not greater than three times its least width and the projection on any side from the face of the supported member shall be not greater than one-half the depth. The depth of a pedestal whose sides are sloped or stepped shall not exceed three times the least width or diameter of the section midway between the top and bottom. A pedestal footing supported directly on piles shall have a mat of reinforcing bars having a cross-sectional area of not less than 0.20 sq. in. per foot in each direction, placed 3 in. above the top of the piles.

DISCUSSION.

Mr. Godfrey.

EDWARD GODFREY (*By Letter*).—The fact that a rule of design has been repeated in regulations and books many times and for many years does not establish that rule as correct nor safe. And a rule of design that has been assailed in all of that period and not defended cannot be considered a sound rule of design.

These regulations contain rules of design that are in the class just stated, rules that for twenty years I have assailed. They are rules that many designers have already abandoned and some designers are afraid to use. I refer specifically to the rules concerning stirrups or short shear members and concerning rodded columns or columns with lateral ties. Stirruped beams and rodded columns are both moribund types of design. Careful designers avoid both. Builders are beginning to fight shy of constructing buildings where dependence is placed on stirrups for shear and rodded columns for support of loads.

It is encumbent on those who repeat rules of design that involve stirrups and rodded columns to produce some better reason for doing so than the mere fact that former regulations included the same.

.On p. 662 the regulations state "Bent-up bars and stirrups shall be considered effective in reinforcing the web only within the area between two vertical planes distant $s/2$ in either direction from the point where the bent-up bar crosses the mid-depth of the beam."

Assume for the sake of argument that this is correct. The distance s is the horizontal spacing of the stirrups. Suppose now that the spacing is made the smallest amount greater than s. This small distance has no web reinforcement. It may be just enough for a crack to form and failure to take place. Does anyone imagine or contend that spacing the stirrups this infinitesimal distance closer marks the difference between failure and safety?

It is just the contention that I have held to for many years, namely, that between stirrups there is no shear reinforcement whatever. Inadvertently the committee here sets forth exactly my contention.

By what rule of stability is the sphere of influence, of a vertical stirrup for example, just equal to the calculated spacing? How can that stirrup reinforce for shear a section of the beam that it does not touch? The same old defenseless rule of spacing stirrups is given in formula (16) carefully disguised in a new mathematical suit. This does not cover up the fact that if, for example, three stirrups are needed to take the shear only one of the three will touch the support, if they are at 45 deg. and none of them will so much as closely approach the support if they are vertical. And it is the support that must ultimately carry all of the

shear. How does it do it? How else, in this standard of design, than by compelling the concrete, unaided by any stirrups to carry it all. This cannot be gainsaid for no web reinforcement whatever is required to cross the section of greatest shear, namely, a section at the face of the support.

W. A. Slater has recently published a paper* which gives the result of tests on shear reinforcement. The thing tested is a thin concrete web reinforced with a dense mesh of reinforcing rods. This thin web is framed in what is, for the size of the member, an enormously heavy frame very heavily reinforced with anchored steel rods. This frame has great rigidity, as proven by results in cases where there is no concrete web, and it is impossible to evaluate the rigidity contributed by the said frame when its four sides act together by reason of the introduction of a concrete web. The whole frame is totally unlike any structural design that could be made use of. Furthermore the stirrups pass from the flanges of these frames (which are tested as beams) to the great end posts, thus being anchored over the supports, a condition never met with in actual stirrup design.

It is through the medium of these impossible tests of types of beams, that could never be duplicated in practice, that attempts are made to put life into a dying system of design. I have not seen for many years a Kahn bar, though standard works hold this out as a typical form of shear reinforcement. But I have seen hundreds of designs of reinforced-concrete beams, designed by the owners of the Kahn bar, where bent-up and anchored rods and no short shear members whatever are used.

The idea that a stirrup is anchored when it is bent over a horizontal rod or is bent into a hook has never been shown to have any meaning. What is it anchored to? Just a little surrounding concrete that has no anchorage to anything else: or it hangs on a horizontal rod that cannot carry vertical loads. This little self-contained rod and its surrounding concrete can perform no function except to tie that small portion of the beam into a unit. Failure in shear can occur and has occurred in hundreds of cases without disturbing the stirrups. A reinforcement about which this can be said is not worthy the name of shear reinforcement.

An upright rod in a column does not reinforce the column unless that rod is itself held upright, and it must be held by something else than the concrete which it is supposed to reinforce. Columns with lateral ties, or rodded columns, as described in these regulations, are not properly reinforced. This has been proven by laboratory tests. It has been proven by hundreds upon hundreds of columns reinforced in this manner which have ingloriously failed in the many reinforced-concrete wrecks.

Other engineers besides myself are beginning to see this. Prof. George F. Swain of the Harvard Engineering School is pointing it out to the profession, as I have been doing for more than twenty years. The reinforced-concrete column is the most important engineering problem before reinforced-concrete engineers today and it will never be solved by writing

* Technologic Paper 314, U. S. Bureau of Standards.

and rewriting the same old fallacies. Slender upright rods cannot be adequately held rigidly by a concrete shaft which these same rods are supposed to bolster up. These rods are themselves under heavy compression due to the shrinkage of the concrete in setting, even before any load is placed upon the column; and when load comes on the column, and the column shortens, more compression must come on the rods. The shrinkage may even be enough to rupture the concrete, if the steel area is large enough, and then we have the absurd structural member consisting of a slender set of rods on which is hanging ruptured concrete.

It is a significant fact that, though Professor Swain's paper read some months ago before the Boston Society of Civil Engineers was abstracted in *Engineering News-Record*, Dec. 23, 1926, no engineer has ventured to comment on the paper in the columns of the *News-Record* and to defend the rodded column. Professor Swain demonstrated the inadequacy of this type of column. If it is a fit type of column to use why does it not find some defender when it has thus been publicly assailed by an authority such as Professor Swain?

In two respects the rules for designing column footings are incorrect and cannot be defended on the basis of engineering analysis. The bending moment at the critical section of a square footing supporting a concentric square column is not that produced by the load on the trapezoid between the diagonals, as stated. Instead of this trapezoid it is the complete rectangle. No analysis can show anything else. It is this blind type of analysis that has helped to place reinforced concrete on the unsound basis so forcibly pointed out by Professor Swain in the paper already referred to. The flat slab analysis is of the same character.

The other fallacy in the footing analysis is equal distribution of rods across the section and out to the unsupported edge. This is a still greater fallacy than the other, for it concentrates load out to the edge of the footing and makes no provision whatever to take the added bending moment.

Steel designers, in designing footings, take the full upward load into one set of I-beams and this load is delivered to an upper layer of I-beams, which in turn deliver their load to the column. The analysis is complete and unassailable. Reinforced-concrete designers would do well to take a lesson from steel designers and analyze structures on engineering principles and not on the basis of a few laboratory tests where tensile strength of the concrete is the predominant factor.

Mr. McMillan. F. R. MCMILLAN.—This report was first presented to the Concrete Institute in 1925. It was not available in time to have preprints at the meeting. It did not appear in print until the 1925 *Proceedings*. Since that date the report has been before the convention of the Institute for discussion and study. At the time the report was presented, there was considerable discussion on the floor of the convention over the provisions of the Joint Committee report, and some of those in this report which were of the same import. Following that discussion there has been no important discussion submitted to the committee from members or outsiders. The

report was prepared for submission as a tentative report at this convention. Now the committee has in mind some revision to which its members were unable to give consideration owing to many other interests, and so we present this report, not as a perfect expression of our opinion or desires, but as a report which we believe will elicit more discussion in the form of a tentative standard than it will if just left before the convention as a progress report. We are aware of many deficiencies, and imperfections, some of which we intend fully to change during the coming year before submitting it to the next convention for adoption as a standard.

One evidence of the necessity for further consideration is the fact that during the last four days since this report has been in the hands of members, we have received more communications than during the entire two years previous to that. In presenting this as a tentative standard, we ask your indulgence to the extent that you do not consider it our final word as a proposed basis for building regulations, but in the nature of a tentative draft submitted for full consideration and discussion. The committee desires, before presenting this for adoption, to include blast-furnace slag among the aggregates referred to in Section B-1, and to include a definition of blast-furnace slag. I, therefore, move to add the words "blast-furnace slag" after the words "crushed stone" in the definition of aggregate in Section B-1, and to include in the definitions, the definition of blast-furnace slag given in the 1926 report of Committee E-5 on Aggregates. I do not have that draft before me, but the definition has been accepted.

Mr. Shepherd.

J. C. SHEPHERD.—I feel that we would make a serious mistake if we adopt this report, including as it does, paragragh C-7, for two reasons; I feel that it limits individuality in engineering, and second, it prohibits the use of a material that has been in general use for years and which has been found to be satisfactory. I refer to the part which says, "Hard-grade billet steel meeting the requirements of the above specification (A-15-14) or rail steel meeting the requirements for rail steel concrete reinforcement bars (Serial Designation A-16-14), of the American Society for Testing Materials, may be used for bars ¾ in. in size and smaller, or for larger sizes where no bending is required." I do not think it is necessary to state that both hard-grade billet steel and rail steel have been used very successfully and quite extensively, and I would recommend the elimination of paragraph C-7 and the substitution of the following: "*Metal Reinforcement.* Metal reinforcement shall conform to the standard specifications of the American Society for Testing Materials, Serial designation A-15-14, covering billet steel concrete reinforcement bars, and Serial designation A-16-14 covering rail steel concrete reinforcement bars. The provision in these specifications for machining deformed bars before testing shall be eliminated."

Mr. Carswell.

J. B. CARSWELL.—After reading over this report I was not at all surprised to hear Mr. McMillan say that he had had quite a discussion and that his correspondence had been heavy on the report. I would venture to suggest that the particular clause just referred to by Mr. Shepherd probably initiated a large proportion of this correspondence.

The first thing that struck me in reading the report was the opening statement to the effect that the American Concrete Institute is an organization of voluntary workers for the most intellectual and economical use of concrete.

The paragraph we are discussing appears to me to be entirely foreign to and out of place in a document of this kind, and certainly if it stays in and becomes operative, it will not be conducive to economy—quite the reverse, and I further submit its isolated presence does not reflect any great intelligence. I have no doubt that the men who passed on this tentative report (precisely eleven in number) did so in the very best of faith, but I am sure that the practical effect of this limiting note has not been carefully considered. Talking as a designing engineer (and I believe most designing men will agree with me), if a restriction is placed on the larger sizes of hard billet bars and rail steel bars, it will almost automatically eliminate these—the most economical bars from our specifications, for the simple reason that an engineer is not going to split his materials. It would be bad design to specify mild steel in one part of the structure and hard-grade steel in another part, and we all know that any job worth calling a job always has ⅞-in. bars and larger in the schedule. The tendency on the other hand, both in this country and in Canada, is toward higher elastic limits. I think I am correct in saying that we have come to the point where it is more or less accepted that higher elastic limits can be used in reinforced concrete than in structural steel. In other words, we need not, in reinforced concrete, sacrifice so much strength to ductility as we have to do in structural steel designs. By introducing a paragraph of this kind, you are more or less stopping the wheels of progress, you are retarding the effort of the engineers along the very paths in which they are working.

I would like also to point out that this clause is contrary and quite foreign to the atmosphere and policy of the specifications of the American Society for Testing Materials which have been in vogue ten or eleven years. It is contrary to the Joint Committee's report, and it is quite contrary to the federal building code report. In fact, the federal building code report, I believe, makes a statement that hard-grade billet bars and rail steel bars are perfectly safe in reinforced-concrete work. Not only so, but it is contrary to the practice in Canada. A paragraph of this kind appearing in the *Proceedings* of the American Concrete Institute would have a strong bearing on what was done in Canada. I would, therefore, suggest that limitation of bar sizes on hard-grade steel and rail steel be eliminated.

Mr. McMillan. MR. MCMILLAN.—I would like to correct the impression that this slipped by accidentally. It was deliberately studied and for many hours. Might we (the committee) point out that this report treats the matter regarding the merits of billet steel and rail steel somewhat differently from any other report. We have placed the rail steel on exactly the same basis as hard-grade billet steel. No matter which side of the argument your

prejudices or opinions are on, you will, no doubt, agree that if there is anything in the argument against the rail steel or the hard-grade billet steel, it is in reference to the bent bars ¾ in. in size or larger. So will you give us credit at least for making the best statement of the problem that has yet appeared in a proposed specification? May I say also that this does not in any way controvert the recognized specifications of the American Society for Testing Materials. Those specifications refer to quality of material only and do not deal with matters of design. This is a matter of reinforced-concrete design.

A. R. LORD.—This discrimination between the bars, ¼ in. and under, and those of larger sizes, is based on the experience of breaking in the field. It has been the experience of quite a number of the members of the committee that bars either of hard-grade billet steel or of rail steel—in other words, of any material which has a high yield point and a high elastic limit—are subject to peculiarly high breakage in handling and bending. This specification simply eliminates bars of such material of ⅞ in. and over which are to be bent, and requires that they be of other material, which does not have so high a yield point. It is not the intention of the committee to discriminate in any way against rail steel; the provision is simply based on the physical qualities of the steel. It is common in current design to have joists with only two bars in them. If one of those bars, the bent bar, which is commonly the larger one, is broken in bending, it is a pretty serious matter in the strength of the structure. Mr. Lord.

MR. CHAPMAN.—Would such a bar meet the requirements of the A. S. T. M. specification? Mr. Chapman.

MR. LORD.—I think they meet them when tested in the standard way, but they do not meet them when delivered on the job bent in the usual commercial way. Mr. Lord.

E. C. HARDING.—About four or five years ago it was our experience that the re-rolled rail or hard billet steel had a tendency to show signs of breakage, not only in the larger but in the smaller bars. Since then, however, we have used quite a tonnage of this grade of steel. It has not been brought to my attention that there has been any breakage or signs of breakage at the bends of this grade of steel. Mr. Harding.

AMERICAN CONCRETE INSTITUTE STANDARD.

Tentative Specifications for Concrete Curb and Concrete Curb and Gutter.*

As Submitted by Committee 8-6.

(Serial Designation S-6E-27T)

I.—General.

Description. The curb, or combined curb and gutter shall be constructed of portland cement concrete in accordance with these specifications and shall comply with the requirements and dimensions shown on the plans.

Subgrade. *Preparation.*—The excavation for curbs shall be of sufficient width and depth as to permit the proper placing of forms, tile where necessary, and the required thickness of foundation.

All soft and spongy materials shall be removed and all depressions filled with suitable material which shall be thoroughly compacted in layers not exceeding four (4) in. in thickness. While compacting the sub-base, the material shall be kept thoroughly wet and shall be in that condition when the concrete is deposited.

Drainage.—When required, a suitable drainage system shall be installed and connected with sewers or other drains indicated by the engineer. The size and requirements for drain tile shall be as shown on the plans or as specified by the engineer.

II.—Materials.

Cement. The cement used shall be a standard brand of portland cement and shall meet the requirements of the American Society for Testing Materials (Serial Designation C9-26) with all subsequent revisions and corrections.

Aggregates. *Fine Aggregate.*—Fine aggregate shall consist of natural sand, or when the engineer so directs, stone screenings, slag sand, tailings, chatts, or other inert materials with similar characteristics, or a combination thereof, having clean, hard, strong, durable, uncoated grains. When incorporated in the mixture, fine aggregate shall be free from frost, frozen lumps, injurious amounts of dust, mica, soft or flaky particles, shale, alkali, organic matter, loam or other deleterious substances. It shall be uniformly graded between the one-fourth (¼) in. and 100-mesh sieves. Not more than 25 per cent of the material shall pass a 50-mesh sieve, and

*NOTE—There is a strong feeling among many engineers that combined curb and gutter construction should be discontinued, but inasmuch as it is still common practice it is included in these specifications. However, separate curb or integral curb is recommended in preference to combined curb and gutter.

not more than 3 per cent of dry material, by weight, shall pass a 100-mesh sieve.

Coarse Aggregate.—Coarse aggregate shall consist of crushed rock, pebbles (gravel), chatts or tailings, or a combination of these materials. The particles shall be clean, hard, tough, durable material, free from vegetable or other deleterious substances, and shall contain no soft, flat, or elongated pieces. The coarse aggregate shall be uniformly graded from fine to coarse with 100 per cent passing the 1½-in. sieve and not more than 5 per cent passing the ¼-in. sieve.

If blast-furnace slag is used for coarse aggregate, it shall be air cooled and exposed to the weather for a period of at least six months prior to use. It shall contain not more than 2 per cent of sulphur and dried slag when shaken to refusal shall have a weight of not less than 70 lb. per cu. ft. In general, slag shall conform to the specifications for "Coarse Aggregate."

Natural Mixed Aggregates.—Natural mixed, or pit-run aggregates shall not be used as they come from the pit, but shall be screened and remixed to agree with the proportions specified. In any case, the quality and grading of the fine and coarse aggregates separated by screening, shall be at least equivalent to that specified in the foregoing paragraphs for fine and coarse aggregates.*

Mixing Water.—The mixing water used shall be clean and free from injurious amounts of oil, acid, alkali or vegetable matter. Ordinarily, any potable water will meet these requirements. **Water.**

Coloring.—If artificial coloring is used in the concrete curb it shall consist only of permanent mineral oxides that are fully guaranteed by the manufacturer to be unaffected by lime, cement or weathering. **Coloring.**

Joint Filler.—The material for filling joints in concrete curbs and gutters shall be a premolded bituminous mastic or suitable waterproof compound that will not become soft and run out in hot weather nor become hard and brittle and chip out in cold weather.

Forms.—All forms shall be of metal, straight, free from warp, and of sufficient strength to remain in alignment and sustain the pressure of the concrete. Forms must be kept clean, oiled or greased and in good condition. **Forms.**

Division plates shall be of ⅛-in. steel, oiled and cleaned each time prior to use.

III.—CONSTRUCTION.

Dimensions.—The cross-section for a separate curb shall be similar to that shown in Fig. 1. **Separate Curb.**

The back shall be straight and perpendicular and the street side shall

*NOTE—In many cases it will be necessary for the engineer to specify the sizes, grading and quality of coarse aggregate in accordance with local conditions. In every case the engineer should provide specifications which will insure the use of the best coarse aggregate which is economically available.

have a batter of approximately one (1) to four (4). The thickness at the base shall be not less than 12 in. and not more than 6 in. at the top.

Combined Curb and Gutter.

Dimensions.—The cross-section of a combined curb and gutter shall be similar to that shown in Fig. 2.

The top of the curb shall be six (6) in. above the gutter line, but this design may be varied according to the locality and drainage facilities.

The combined curb and gutter shall be divided into sections not more than 10 ft. long. The construction joint must insure complete separation of the sections.

At street intersections where two curbs meet, the corner shall be rounded to a radius of not less than 20 ft.

Expansion Joints.

Expansion Joints.—Expansion joints shall be required at each curb return, and at such intervals as defined by the engineer or as shown on

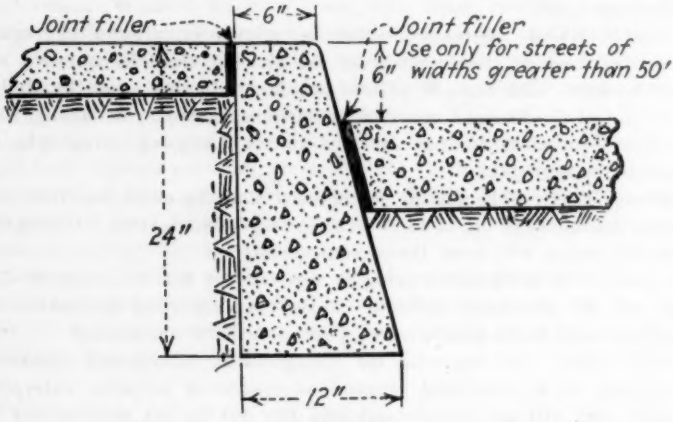

FIG. 1

the plans. Unless otherwise designated on the plans the expansion joint shall be one-half (½) in. in thickness and of a material as defined in these specifications under MATERIALS—*Joint Filler.*

Proportions.

Proportions.—The proportions of aggregate to cement shall be such as to produce concrete that can be puddled readily into the corners and angles of the form without excessive spading and without undue accumulation of water or laitance on the surface. The proportion of fine and coarse aggregates shall be such that the ratio of the coarse to the fine shall not be less than 1 nor more than 2, nor shall the amount of coarse material be such as to produce harshness in placing or honeycombing. Proportions approximating a 1: 2: 3 mix are recommended.

Measuring Materials.

Measuring Materials.—The method of measuring materials, including water shall be one which will insure separate accurate proportions at all times. A bag of portland cement (94 lb. net) shall be considered as one (1) cu. ft.

Machine Mixing.—The concrete shall be mixed in a batch mixer of such capacity that only whole bags of cement are used in each batch. It shall be a type approved by the engineer. Mixing.

Mixing shall continue for at least 1 min., but preferably 1½ mins. after all materials, including water, are placed in the drum, and before any part of the batch is discharged.

Hand Mixing.—Mixing by hand shall only be permitted on jobs that are too small to justify the use of a machine mixer. When it is necessary to mix by hand, the mixing shall be done on a water-tight platform and the materials shall be turned at least three times and until the mass is a uniform color and all the particles coated with a cement mortar.

Central Mixing.—The use of central mixing plants, and the transportation of mixed concrete is permitted under these specifications, provided there is no segregation of the mixed concrete when it is delivered at the point where it is to be deposited. The concrete must be of a workable consistency when placed. Under no circumstances shall concrete,

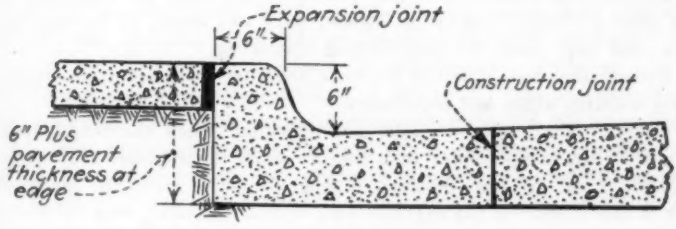

FIG. 2

mixed at a central plant, be deposited directly into the forms, but shall be rehandled from a platform, sled, trough or other suitable device which can travel alongside the curb forms.

Retempering.—Concrete which has partially set shall not be retempered by being mixed with additional materials or water.

Consistency.—The concrete mixture shall contain no more water than is necessary to produce a workable mass which can be brought to a satisfactory finish. The amount of water used shall not exceed 6¼ gal. per sack of cement. Consistency.

Placing.—After mixing, the concrete shall be placed in the forms in separate layers four or five inches thick until the forms are completely filled. Each layer shall be thoroughly tamped and spaded before the next is added. Particular attention shall be given to spading the concrete close to the forms in order to prevent honeycombing and so that the exposed surface may have a smooth uniform appearance. Placing.

When the forms are completely filled the concrete shall be struck off with a templet cut to the desired curb design. The concrete shall then be finished with a wooden float, in a manner which will thoroughly com- Finishing.

pact the mass and leave a true even surface. When required, the surface may be troweled smooth, but excessive working with a steel trowel should be avoided. The upper edges of the curb shall be rounded with an edging tool.

The forms shall be removed within twenty-four (24) hours after the concrete has been placed and minor defects filled with mortar composed of one (1) part of portland cement and two (2) parts of fine aggregate. The front or face forms shall be removed within twelve (12) hours after concrete has been placed, and the top and face of the curbing, from the top to eight (8) in. below shall be finished while the concrete is still green by frequently wetting a soft brick or a wood block and rubbing the surface until it is smooth. Within twenty-four (24) hours the curbing shall be rubbed once more with a block and water as before to produce a smooth surface. Brush finishing or plastering shall not be permitted on the top or the face of the curbing. All separator joints shall be opened from top to bottom, the edges adjacent to the joint being sharp and clean cut.

Curing. *Curing and Protection.*—The concrete curb shall be kept wet for 7 days after placing. It may be covered with wet earth, or burlap, hay or straw, and kept moist by intermittent sprinkling.

Protection. During the curing period the curb shall be protected from injury by proper warning signs and barricades.

Cold Weather Work. Concrete shall not be mixed nor deposited when the temperature is below freezing, except under such conditions as the engineer may direct in writing. If, at any time during the progress of the work, the temperature is, or in the opinion of the engineer will, within 24 hours, drop to 38 deg. F. the water and aggregates shall be heated, and precautions taken to protect the concrete from freezing until at least 10 days old. In no case shall concrete be deposited upon a frozen subgrade, nor shall frozen materials be used in the concrete.

W. M. ACHESON, *Chairman*,
C. R. EGE, *Secretary*.

Standard Building Units.

As Submitted by Committee P-1.

Committee P-1 has been appointed and is acting as the standing committee on standard sizes of concrete building block, brick and tile of the Division of Simplified Practice, U. S. Department of Commerce. On the basis that the types of machines now being sold define very closely the sizes of these concrete products which would be made in the future, a canvass has been made of the leading manufacturers of concrete products machinery. From them the percentage of machinery sold during the past year making concrete block, brick or tile in accordance with the sizes adopted by the Division of Simplified Practice has been obtained. The results of this canvass were extremely satisfactory. Fourteen out of fifteen leading manufacturers replied that practically 100 per cent of the machines sold by them make units of standard sizes. In the exceptional case, the units produced were of standard width and length, differing only in height. Our committee therefore reports that progress toward standardization of sizes for concrete masonry units have been very rapid and recommends to the Division of Simplified Practice, U. S. Department of Commerce, that no changes be made in their standard sizes for these units during the ensuing year.

A series of tests was made under the auspices of this committee to study the effect of various gradings of aggregates having the same fineness modulus upon the strength and absorption of tamped concrete building units. The program was made possible through the co-operation of Prof. R. L. Morrison, director, state highway laboratory, Ann Arbor, Mich., who made all the laboratory tests, and E. W. Dienhart, who placed the plant of the Acme Concrete Products and Gravel Co., Cement City, Mich., at the disposal of the committee. A discussion of the tests is appended to this report and is a portion thereof.

Sanitary engineers have frequently requested specifications covering the proper physical characteristics of concrete manhole and catchbasin block. The use of this type of concrete unit is becoming more and more extensive. For example, over 5,000,000 were used in Cook County, Ill., during the past year. In so much as concrete manhole and catchbasin block are similar to concrete building block in their general characteristics and method of manufacture, the Board of Direction assigned committee P-1 to formulate specifications. These are herein included, which your committee recommends be adopted as tentative standards.

At the last convention, amendments to the Proposed Standard Specifications for Concrete Building Block, Concrete Building Tile and Concrete Brick* were adopted which clarified the wording requiring specimens to be in a dry state for compression tests. Other amendments were tentatively adopted to specify in more detail methods for making compression tests. We recommend that these latter amendments now be adopted as standard. In order that the membership may be familiar with the specifications after all of these changes have been made, copies are included as a portion of this report.

E. W. DIENHART, *Chairman.*
C. L. BOURNE, *Secretary.*

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* Note: The proposed Standard Specifications for Concrete Brick, adopted in February, 1926, as tentative standards, were continued as tentative standards, by vote of Convention, Feb. 22, 1927, for another year in the hope that cooperation with a committee of the American Society for Testing Materials may result in the adoption of a single standard for brick.

Report of Tests on Concrete Building Tile.

Cement City, Michigan.

Submitted by Committee P-1.

The purpose of these tests was to study the effect of various gradings of aggregate having the same fineness moduli on the strength and absorption of tamped concrete building tile. The tests were made under the auspices of Committee P-1, American Concrete Institute. Acknowledgment is made to Prof. R. L. Morrison who tested the tile and the cement at the Michigan State Highway Laboratory, Ann Arbor, and to E. W. Dienhart, who furnished materials and the plant of Acme Concrete Products and Gravel Co., Cement City, Mich., where the specimens were made by C. L. Bourne, representing the committee.

Although the number of variables was somewhat limited, it is felt that the results indicate tendencies which are of practical value to the products manufacturer.

Outline of Tests.

Compression and absorption tests on standard 5 x 8 x 12-in. concrete building tile were made from 16 different batches of concrete with the following variables:

(a) Four sands from the same pit ranging in fineness modulus from 2.00 to 3.50, mixed with pebbles passing a ⅝-in. sieve having a fineness modulus of 6.16 to give fineness moduli of mixed aggregate of 3.50 and 4.

(b) 1 : 3 and 1 : 5 mixes by volume were used for each of the above aggregate gradings.

Materials.

1. *Aggregates.*—The four sands were taken from different parts of the same pit of the Acme Products and Gravel Co., Cement City, Mich. Pebbles passing a ⅝-in. screen were taken from the Company's stockpile.

2. *Cement.*—Portland cement having the following characteristics was used.

Soundness	O. K.
Initial setting	4 hr. 5 min.
Final setting	6 hr. 20 min.
Retained on 200-mesh sieve	13.9%
Tensile strength 1 : 3 standard briquets 7 days	330 lb. per sq. in.

3. *Water.*—The amount of water used was the maximum practical under the operating conditions. This amount is less than that required to obtain concrete of maximum strength and therefore any reduction in the water content would decrease strength. The result of insufficient water is shown in discussion of Batch 1 in the third conclusion.

FIELD OPERATIONS.

The tile were 5 x 8 x 12-in. and were mechanically tamped in a machine of the stripper type. The regular operator of the plant handled the machine and each tile was given the same number of tamps.

Mixtures are given as volume of cement to volume of dry rodded mixed aggregate. Directly after the mixtures were determined, the aggregates were measured accurately and placed in heavy paper sacks in the cement storage room of the plant until used.

Each batch required one bag of cement. Aggregates were measured in a 1-cu. ft. box using a rule graduated to 1/100 of a foot for measuring fractional cubic feet. Consistency was judged by taking a handful of concrete and rubbing on a smooth metal surface. Observations taken of tile while being stripped from the machine showed a good uniformity in moisture appearing on the surface, except for Batch No. 1 which was a little drier than subsequent batches. This was due to the fineness of the aggregate and possibly the dry condition of the chain conveyor, belt conveyor, and machine hopper. A new batch was not begun until the previous one was completely used and the machine hopper cleaned.

The concrete was mixed one minute dry and 3 minutes after water had been added. Concrete from the mixer was carried to the tile machine by a chain conveyor (22 ft. long) and a belt conveyor (6 ft. long).

The tile were placed on racks on a curing room as soon as made. The first four and last two tile from each batch were discarded. This left five or six tile to be tested from each batch. Curing consisted of sprinkling as soon as possible and 3 times daily thereafter for 5 days. No artificial heat was used. After remaining in the curing room for 3 days, the tile were stored in an enclosed shed until shipped to the laboratory. This precaution was taken so that all specimens would receive the same curing.

CONCLUSIONS.

1. In these tests, aggregates having the same fineness moduli produced building tile of essentially the same strength with other manufacturing details constant. Identical fineness moduli were obtained through the use of a coarse aggregate combined with four different sands with fineness moduli varying from 2.00 to 3.50. In all cases, the 28-day strength of the building tile varied from the average 15 per cent or less with the same fineness modulus of different gradings for the same mixture. The variation is small considering that only one batch was used to represent each condition.

2. In general, building tile having the greater compressive strength was less absorbent and was composed of concrete having a higher weight per cubic foot.

3. The necessity for closely regulating the amount of water used in manufacture of dry tamped products is shown by Batch 1. This batch produced tile of good appearance, but was somewhat drier than subsequent

batches, resulting in a reduction of strength of 40 per cent from the average of the other tile in this group, with an increase of absorption of 1.8 per cent. The reason for this discussed under the subject of "Materials."

TABLE 1—SIEVE ANALYSES OF AGGREGATES

Tyler Standard screen scale sieves. Fineness modulus (F. M.) is sum of percentages retained on the sieves, divided by 100.

Aggregate	Per Cent Coarser than Sieve								F. M.	Weight of Aggregate, lb. per cu. ft.		Moisture, per cent by weight
	100	48	28	14	8	4	⅜	¾		Damp and Loose	Dry and Rodded	
Sand 0	96	85	15	3	1	0	0	0	2.00	81.5	102.5	4.2
" 1	97	88	34	14	4	0	0	0	2.37	89.0	109.0	3.6
" 2	99	92	51	31	16	3	0	0	2.92	91.4	111.0	3.1
" 3	99	96	68	50	32	6	0	0	3.50	94.0	112.0	3.8
Pebbles	100	100	100	100	99	98	19	0	6.16	97.0	105.0	2.6

TABLE 2—COMPRESSION TESTS OF CONCRETE BUILDING TILE

Concrete Tile: Standard 5 x 8 x 12-in. made on stripper machine by regular plant operator. *Mix:* By volume of dry rodded combined aggregates. Concrete mixed in paddle type batch mixer. *Consistency:* Wet as possible without slumping. *Cured:* Moist 5 days, then in air. *Strengths* are the results of 5 or 6 tests from the same batch computed on basis of gross area. *Absorptions* are the results of 2 tests. *Age at Test:* 30 days for both compression and absorption.

Batch No.	Aggregates				Mixed Aggregate				Absorption, per cent by weight	Weight of Dried Tile		Compressive Strength, lb. per sq. in.
	Sand		Pebbles		Sand, per cent	Pebbles, per cent	Weight, lb. per cu. ft.	F. M.		lb.	lb. per cu. ft.	
	No.	F. M.	Size	F. M.								
1:3 Mix												
1*	0	2.00	⅛–⅜	6.16	64	36	119.0	3.5	9.0*	16.8*	122.0*	780*
2	1	2.37	"	"	70	30	121.5		6.8	18.0	134.0	1180
3	2	2.92	"	"	82	18	119.0		7.4	18.0	133.1	1230
4	3	3.50	"	"	100	0	112.0		7.1	18.3	137.1	1500
Average*									7.2	18.1	134.7	1300
5	0	2.00	"	"	52	48	122.0	4.0	6.1	18.6	135.2	1280
6	1	2.37	"	"	58	42	125.0		5.8	18.5	136.5	1350
7	2	2.92	"	"	67	33	121.0		5.7	19.0	141.4	1360
8	3	3.50	"	"	82	18	119.5		6.0	18.9	134.3	1310
Average									5.9	18.7	136.8	1320
1:5 Mix												
9	0	2.00	⅛–⅜	6.16	64	36	119.0	3.5	8.4	17.5	124.8	890
10	1	2.37	"	"	70	30	121.5		8.0	17.6	129.2	800
11	2	2.92	"	"	82	18	119.0		8.1	17.3	129.8	990
12	3	3.50	"	"	100	0	112.0		7.6	17.2	123.7	800
Average									8.0	17.4	126.9	870
13	0	2.00	"	"	52	48	122.0	4.0	6.6	17.5	126.9	770
14	1	2.37	"	"	58	42	125.0		6.3	18.1	128.5	940
15	2	2.92	"	"	67	33	121.0		6.7	17.7	130.7	930
16	3	3.50	"	"	82	18	119.5		6.2	18.3	130.1	970
Average									6.5	17.9	130.1	900

* Not included in average; batch too dry, as explained in text.

AMERICAN CONCRETE INSTITUTE STANDARD.

TENTATIVE SPECIFICATIONS FOR CONCRETE MANHOLE AND CATCH BASIN BLOCK.*

Submitted by Committee P-1, Standard Building Units.

(Serial Designation P-1C-27T)

I.—GENERAL.

1. The purpose of these specifications is to define the requirements for solid concrete manhole and catch-basin block.

2. The word "concrete" shall be understood to mean portland cement concrete.

Strength Requirements.

3. The ultimate compressive strength of solid concrete manhole and catch-basin block at 28 days of age or when delivered upon construction work shall average not less than 1,500 lb. per sq. in. of cross-sectional area of the unit as laid in the wall. No individual unit shall test less than 1,200 lb. per sq. in. of cross-sectional area under the same conditions.

Absorption Requirements.

4. Concrete manhole and catch-basin block shall not absorb more than 8 per cent of the dry weight of the unit when tested as hereinafter specified. No individual unit shall absorb more than 10 per cent of the dry weight of the unit.

Sampling.

5. Specimens for tests shall be representative of the commercial product of the plant.

Not less than three and preferably five specimens shall be provided for each test.

6. Specimens used in the absorption test may be used for the strength test.

II.—METHODS OF TESTING.

Absorption Test.

7. The specimens as received shall be immersed in clean water at approximately 70 deg. F. for a period of 24 hours. They shall then be removed, the surface water wiped off, and the specimens weighed. Specimens shall be dried to a constant weight at a temperature of from 212 deg. to 250 deg. F. and reweighed. Absorption is the difference in weight divided by the weight of the dry specimens and multiplied by 100. The average of the tests of not less than three specimens shall be taken as the absorption.

Strength Test.

8. Specimens for the strength test shall be dried to constant weight at a temperature of from 212 deg. to 250 deg. F.

9. The specimens to be tested shall be carefully measured for overall dimensions of length, width and height.

10. Bearing surfaces shall be made plane by capping with the plaster of paris or a mixture of portland cement and plaster which shall be

* Note: Adopted as tentative standard by Convention, Feb. 22, 1927.

allowed thoroughly to harden (from 3 to 6 hours) before the test. No point on the surface shall deviate from the plane more than 0.003 in. The cap shall not be thicker than ⅛-in. and shall be formed by means of a cast-iron or steel plate or plate glass which has a true surface.

11. Specimens shall be accurately centered in the testing machine.

12. The load shall be applied through a spherical bearing block placed on top of the specimen. The rate of loading after 50 per cent of the ultimate load has been applied shall be not greater than that which will produce a shortening of the specimen of 0.02 in. per minute.

13. When testing other than rectangular block or tile care must be taken to see that the load is applied through the center of gravity of the specimen.

14. Machined steel or cast-iron plates of sufficient thickness to prevent appreciable bending shall be placed between the spherical bearing block and the specimen. In no case shall the distance between the edge of the spherical bearing block and the end of the bearing plate be greater than twice the thickness of the plate. Where a number of thin plates are used, in no case shall the plates be less than one inch thick nor shall any plate extend beyond the one immediately above it a greater distance than twice the thickness of the plate.

15. The specimen shall be loaded to failure.

16. The compressive strength in pounds per square inch of gross cross-sectional area is the total applied load in pounds divided by the cross-sectional area in square inches. The average of the tests of not less than three specimens shall be taken as the comparative strength.

AMERICAN CONCRETE INSTITUTE STANDARD.

STANDARD SPECIFICATIONS FOR CONCRETE BUILDING BLOCK AND CONCRETE BUILDING TILE.*

Submitted by Committee P-1, on Standard Building Units.

(Serial Designation P-1A-27)

I.—GENERAL.

1. The purpose of these specifications is to define the requirements for concrete building block and concrete building tile to be used in construction.

2. The word "concrete" shall be understood to mean portland cement concrete.

Strength Requirements.

3. According to the strength in compression 28 days after being manufactured or when shipped, concrete block and concrete tile shall be classified as heavy load bearing, load bearing, and non-load bearing on the basis of the following requirements:

Name of Classification	Compressive Strength, lb. per sq. in. of gross cross-sectional area as laid in the wall. Aver. of 3 or more units	Min. for Individual Unit
Heavy load bearing block or tile	1200	1000
Medium load bearing block or tile	700	600
Non-load bearing block or tile	250	200

4. The gross cross-sectional area of a one-piece concrete block or tile shall be considered as the product of the length times the width of the unit as laid in the wall. No allowance shall be made for air spaces in hollow units. The gross cross-sectional area of each unit of a two-piece block or tile shall be considered the product of the length of the unit times one-half the thickness of the wall for which the two-piece block or tile is intended.

5. The compressive strength of the concrete in units of all classifications except "non-load bearing block" shall be at least 1,000 lb. per sq. in., when calculated on the minimum cross-sectional area in bearing.

Absorption Requirements.

6. Concrete Building Block and tile to be exposed to soil or weather in the finished work (without stucco, plaster or other suitable protective covering) shall meet the requirements of the absorption test.

7. All concrete building block and tile not covered by Paragraph 6 need not meet an absorption requirement.

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* Adopted as Standard Specifications, Feb. 22, 1926, and referred to letter ballot for final adoption, ballot to be canvassed May 26, 1927.

8. Concrete block and tile shall not absorb more than 10 per cent of the dry weight of the unit when tested as hereinafter specified, except when it is made of concrete weighing less than 140 lb. per cu. ft. For block or tile made with concrete weighing less than 140 lb. per cu. ft. the absorption in per cent by weight shall not be more than 10 multiplied by 140 and divided by the unit weight in pounds per cubic foot of the concrete under consideration.

Sampling.

9. Specimens for tests shall be representative of the commercial product of the plant.

10. Not less than three and preferably five specimens shall be required for each test.

11. The specimens used in the absorption test may be used for the strength test.

II.—METHODS OF TESTING.

Absorption Test.

12. The specimens shall be immersed in clean water at approximately 70 deg. F. for a period of 24 hours. They shall then be removed, the surface water wiped off, and the specimens weighed. Specimens shall be dried to a constant weight at a temperature of from 212 deg. to 250 deg. F. and re-weighed. Absorption is the difference in weight divided by the weight of the dry specimens and multiplied by 100.

Weight of Concrete.

13. The weight per cubic foot of the concrete in a block or tile is the weight of the unit in pounds, divided by its volume in cubic feet. To obtain the volume of the unit fill a vessel with enough water to immerse the specimen. The greatest accuracy will be obtained with the smallest vessel in which the specimen can be immersed with its length vertical. Mark the level of the water, then immerse the saturated specimen and weigh the vessel. Draw the water down to its original level and weigh the vessel again. The difference between the two weights divided by 62.5 equals the volume of the specimen in cubic feet.

Strength Test.

14. Specimens for the strength test shall be dried to constant weight at a temperature of from 212 deg. to 250 deg. F.

15. The specimens to be tested shall be carefully measured for overall dimensions of length, width and height.

16. Bearing surfaces shall be made plane by capping with plaster of paris or a mixture of ½ portland cement and ½ plaster of paris, which shall be allowed thoroughly to harden (from 3 to 6 hours) before the test. No point on the surface shall deviate from the plane more than 0.003 in. The cap shall not be thicker than ⅛-in. It shall be formed by means of an accurately machined metal plate or a heavy plate glass having a true surface.

17. Specimens shall be accurately centered in the testing machine.

18. The load shall be applied through a spherical bearing block placed on top of the specimen. The rate of loading after 50 per cent of the ultimate load has been applied shall not be greater than that which will produce a shortening of the specimen of 0.02 in. per minute.

19. When testing other than rectangular block or tile care must be taken to see that the load is applied through the center of gravity of the specimen.

20. Machined-steel or cast-iron plates of sufficient thickness to prevent appreciable bending shall be placed between the spherical bearing block and the specimen. In no case shall the distance between the edge of the spherical bearing block and the end of the bearing plate be greater than twice the thickness of the plate. Where a number of thin plates are used, in no case shall the plates be less than one inch thick nor shall any plate extend beyond the one immediately above it a greater distance than twice the thickness of the plate.

21. The specimen shall be loaded to failure.

22. The compressive strength in pounds per square inch of gross cross-sectional area is the total applied load in pounds divided by the gross cross-sectional area in square inches.

See Vol. 24, p. 834-5 - To revise requirement for non-load bearing block and tile as follows:

(Tentative Amendment Paragraph 3. Feb. 1928)

Revised Specifications

3. The average compressive strength of 3 or more concrete block or concrete building tile in lb. per sq. in. of gross cross-sectional area as laid in the wall shall not be less than 700 lb., no one unit falling below 600 lb. 28 days after being manufactured or when shipped.

Where ever concrete block or building tile are used to carry unusual heavy loads, the average compressive strength of 3 or more of these units 28 days after being manufactured or when shipped in lb. per sq. in of gross cross-sectional area as laid in the wall shall be at least 10 times the figured superimposed load to be applied.

Non-load-bearing concrete block and concrete tile shall have sufficient strength necessary to prevent excessive breakage during delivery and handling.

DISCUSSION.

CLOYD M. CHAPMAN.—There are two matters I want to put before the Institute. You probably know that the American Society for Testing Materials is formulating standards for brick. That society had a standard, but it is now being revised by the committee on Brick. Would it not be wise to continue our specifications as tentative for another year in the hope that we may have one standard for concrete brick? It would be unfortunate to have two standards and I think this Institute will, through its committee P-1, set up some method of co-operation with the brick committee of the American Society for Testing Materials during the coming year. It seems very likely that by such a procedure there might be evolved a single standard. Mr. Chapman.

The second matter is the Report of Tests on Tile (p. 691). Here we have reported a series of tests made on concrete building tile. It is an interesting report, but this Institute has done much to advance the knowledge of the water-cement ratio and the effect of water on the strength of concrete of all kinds. It is not limited to building concrete, it is just as much a fact that the amount of water used in mixing concrete for a building tile or brick has an effect upon its strength as it is in the case of other concrete, but here we have a report in which the water content is not even mentioned. The report even indicates in one case that the reduction in strength of 40 per cent from the average of the other tile in the group was occasioned by a lower water-cement ratio, which, in the general acceptation of the information adduced, ought to increase its strength. This is easily explainable. This point falls on the part of the water-cement ratio-strength curve where there is too little water present to produce a workable mix. This report ought to include distinct and concise references to the effect of water which, after all, is of first importance. This report would indicate that the factor which had the greatest effect on the strength of concrete was the fineness modulus. I would like to see this portion of the report re-edited by this sub-committee before publication in the *Proceedings*, in order to make clear the reason for this reduction of 40 per cent by a lower water-cement ratio.

ALBERT S. BURNS.—I read the specifications from the standpoint of the testing engineer. In all three of the specifications appears a clause that the plaster cap shall not exceed ⅛ in. in thickness. Evidently the committee that did this work has not had much experience in capping in commercial laboratories. The specimens we receive do not come into the laboratory with the top and bottom in parallel planes, nor with either top or bottom in one plane. Invariably we find one or more corners high; Mr. Burns.

one side of the block is always rough. It is impossible, even though a block is dressed with a file, as is the practice in our laboratory, to get a cap that does not at some point exceed ⅛ in.

So far as I know this is the only national specification covering the testing of concrete blocks, and as such it is bound to receive the recognition of the courts. It places the testing laboratories, particularly municipal ones, in a position where they must adopt this specification because it is the only thing that we have to back us up. I do not believe that in this clause limiting the thickness of a cap to ⅛ in. a specification is made possible of universal use.

Then let us take the case of the cinder block. Cinder blocks come into the laboratory with top or bottom full of holes, most of them ¼ in. and in many cases ½ in deep, a condition rendering it impossible to put on a cap ⅛ in. thick. The proper method of stating this would be to say that the cap should be as thin as possible. It is a practical method and practical problems are the problems of the laboratory.

As to testing dry—this requires laboratories to steam dry these block before test. There is considerable difference in the strength of block between testing dry and wet. About a year and a half ago I conducted a series of tests in the municipal laboratories of Indianapolis to find out what were the facts. I found a variation between a bone-dry condition and a saturated one of as high as 40 per cent, but I do believe that requiring steam drying before testing is erroneous. I believe that room drying, say a loss of 1 per cent in four hours at a temperature above 70, is better.

The other question I wish to bring up is a point of method of tests on brick. In this specification it stipulates that the brick shall be tested flat. This brings us again to the question of two standards. The present standard of the American Society for Testing Materials requires a test of the half brick on edge. It is manifest that results obtained by these two methods are not at all comparable, inasmuch as the two materials are used for the same purpose. The method of testing should be the same.

I want to move you that the clause in all three specifications stating that the thickness of the plaster cap shall not exceed ⅛ in. be removed and in its place there be substituted a statement that the plaster cap shall be "as thin as possible."

Mr. Wilk. BENJAMIN WILK.—I agree with Mr. Chapman that it is not desirable for the Institute to accept a standard specification on concrete brick when the American Society for Testing Material is working on the same subject. Concrete brick is a wall unit the same as a concrete block and the concrete block specifications has a range of from 700 to 1,200 lb. compressive strength per sq. in. of gross cross-sectional area as laid in the wall. Why is it necessary to have a concrete brick of 1,500 lb. when a heavy load-bearing block or tile requirement is only 1,200 lb.?

Mr. Bourne. MR. BOURNE.—In a concrete block or hollow tile the air space requires practically a 1,500-lb. concrete to obtain the 1,000 or 750 lb. requirement. The committee, I believe, decided that the characteristics of a concrete,

more than 1,000 lb., were not good enough for exposure to weather conditions.

MR. WILK.—At Columbia University tests of concrete brick showed an efficiency around 60 per cent to 70 per cent. Some tests I recently completed on concrete brick with as low as 1,200 lb. per sq. in. for individual brick in piers and walls show practically the same efficiencies. After all it is a question of practical use of the material, and that, from the weather standpoint, is covered to a certain extent by absorption. It seems to me to be penalizing the concrete brick when we ask it to be 1,500 lb. per sq. in. of gross area. Mr. Wilk.

MR. CHAPMAN.—The steam-oven drying of building block before testing should be considered from a practical viewpoint. The intention of the specification is to give the block the best chance of passing the test. A block tested dry will test higher than the same block tested wet, but if the block does not need that advantage, if it will pass the specification even in a moist or damp condition as received, why not let it be tested in that condition? It would test higher if it were dry. If a provision were inserted in the specifications which would make the dry test the final or referee test where there was failure under damp or moist conditions, it would meet the objection raised. Better still, test the blocks damp, and if they fall short of the standard, other tests can be made on dry specimens. I do not think that would be a practical thing to include both damp and dry tests in the specification. Mr. Chapman.

MR. BOURNE.—In general, I think a specification should be a standard proposition. This would include a variable in a specification; it would give a chance for misinterpretation and be rather confusing. The intent of the specification naturally is that the block be tested in a dry condition. As shown in last year's committee's report, the tests from various laboratories have been considerably different on specimens even from the same batch. The idea was to make a workable specification, one that would not prove a hardship upon the testing laboratory, but at the same time be uniform. Mr. Bourne.

MR. LAGAARD.—I agree with Mr. Bourne in this respect. The chief advantage of the testing is uniformity; if you test moist you will get variables up to as high as 30 per cent or 40 per cent reduction in strength and unless you specify the conditions of moisture, you are not specifying the strength at all. Mr. Lagaard.

MR. CHAPMAN.—I disagree. The thing we are after is a certain minimum strength; we do not care how much a product goes beyond that minimum strength, we are not after uniformity, we are after a strength that is not below the minimum. We permit two tests of brick in compression—the strength of the bricks are required to be a certain amount at 28 days or when shipped; if the specified strength is developed in say ten days when the brick are shipped, well and good, but the 28-day test is the controlling factor. Mr. Chapman.

MR. WILK.—I think we ought to have a standard so that when a man Mr. Wilk.

is thinking in terms of his block, and he is talking to another man about it and says, "My block is 1,000 lb.," they will both know how the tests are made. If, however, one tests 700 lb. wet and the other tests 1,000 lb. dry, they might be exactly the same block and yet no one would know it. There might be a clause saying that if desired the block can be tested wet.

Mr. Woolson. MR. WOOLSON.—I can see this complication arising, in the practical application of this so far as its use for city requirements is concerned. There may be a demand for quick delivery of block either on account of oncoming cold weather or something of that kind and a man may elect to use a quick-hardening cement. The man who is selling the block wants to meet the city requirements. Why compel him to hold the block longer than is necessary?

AMERICAN CONCRETE INSTITUTE
BUSINESS REPORTS

Annual Report of the Board of Direction to the Members—April 1, 1927.

This volume 23 of the *Proceedings* is the best report of the Institute's technical activities. The "News Letters," seven of which have been published and sent to members since Volume 22 of the *Proceedings* was issued, have supplied timely notice of the work and progress of the organization. The customary annual report of the Board of Direction becomes chiefly a formality which may best be made brief.

In the fiscal year ended June 30, 1926, 522 new active members were added; 129 were lost, a net gain of 393. In that period 11 supporting members were added; 5 were lost, a net gain of 6. The active membership at the close of the fiscal year was 1,819, supporting membership 116, total at the close of the fiscal year 1,935.

Since then, to April 1, 1927, further gains bring the active membership to 2,266, and supporting membership 126—total, 2,392, a net gain of 457 members in the first nine months of the present fiscal year.

Following is a report of the annual audit for the fiscal year ended June 30, 1926:

MEISSNER AUDIT CO.
DETROIT, MICH.

August 12, 1926.

Mr. Harvey Whipple, Treasurer,
American Concrete Institute,
2970 West Grand Boulevard,
Detroit, Michigan.

Dear Sir:

We have conducted a Cash Audit of the American Concrete Institute from July 1, 1925, to June 30, 1926, as shown by the books of the Company, and have found that all Cash reported received has been properly deposited with the National Bank of Commerce of Detroit, Michigan, in the name of The American Concrete Institute, and that withdrawals from this account are covered by proper vouchers.

The Bank Account has been verified, and agrees with Statement furnished by the National Bank of Commerce, as at June 30, 1926.

No attempt has been made to verify cash received, as reported, with your membership.

Thanking you for the assistance and courtesy shown us during the conduct of our Audit, we remain,

Yours respectfully,
MEISSNER AUDIT COMPANY,
By JOHN C. MEISSNER,
Public Accountant.

AMERICAN CONCRETE INSTITUTE.
BALANCE SHEET.
June 30, 1926.

ASSETS.

Cash:		
On Hand	$500.00	
In National Bank of Commerce	5,719.46	
Total Cash		$6,219.46
U. S. Treasury Certificates		4,500.00
Accounts Receivable:		
Active Members (239 @ $10.00 and 1 @ $8.80)	$2,398.80	
Contributing Members (8 @ $50.00)	400.00	
Miscellaneous	939.45	
Total Accounts Receivable		3,738.25
Inventories:		
164—1905-1919—@ $0.50	$82.00	
862—1920-1925—@ 1.00	682.00	
220—1926 —@ 3.00	660.00	
Total Inventories		1,424.00
Total Assets		$15,881.71

LIABILITIES.

Accounts Payable: *Proceedings* Expense	$6,349.90	
Deferred:		
Dues Paid in Advance	90.00	
Reserve:		
For Loss Due to Delinquent Members	2,000.00	
Surplus:		
Deposited by Members	7,441.81	
Total Liabilities		15,881.71

AMERICAN CONCRETE INSTITUTE.
RECEIPTS AND DISBURSEMENTS.
July 1, 1925, to June 30, 1926.

RECEIPTS.

Cash on hand July 1, 1925	$745.60
Dues, Active	16,649.20
Dues, Contributing	5,485.00
Reprint Sales	1,782.96
Proceedings Sales	1,908.49
Interest Earned, Bank Balances	146.22
Interest Earned, Treasury Certificates	143.12
Certificates (Membership A. C. I.)	.45
Total Receipts	$26,861.04

DISBURSEMENTS.

Imprest Cash	$200.00	
Auditing	34.38	
Convention Expense	1,032.19	
Exchange	.40	
Membership, National Fire Protection Ass'n	60.00	
Miscellaneous Contingencies	221.41	
Office Expense	792.70	
Postage	1,267.19	
Reprint Expense	3,260.55	
Printing and Multigraphing, Stationery and Letters	3,003.55	
Proceedings Expense	1,434.46	
Rent	673.87	
Secretary's Bond	50.00	
Salaries	8,737.58	
Traveling	428.23	
	$21,196.51	
Less—Discount Net	54.93	
Total Disbursements		21,141.58
Cash in Bank June 30, 1926		$5,719.46

AMERICAN CONCRETE INSTITUTE.
BANK RECONCILIATION.
June 30, 1926.

Balance as per Ledger	$5,719.46
Add—Checks Outstanding	508.33
Balance per Bank Statement June 30, 1926	$6,227.79

LIST OF REGISTRANTS

LIST OF REGISTRANTS AT 1927 CONVENTION.

Star (*) denotes member.

*ABEL, NORMAN A., Allegheny County, D. P. W., 7722 Brashear St., Pittsburgh, Pa.
ABRAMS, A. J., Barton Spider Web System, Chicago, Ill.
*AFFLECK, B. F., Universal Portland Cement Co., Chicago, Ill.
*AHLERS, JOHN G., Barney Ahlers Construction Co., New York, N. Y.
AKERS, CHAS. W., Nashville Breeko Block & Tile Co., Nashville, Tenn.
*ALBRECHT, R. W., 1991 Port Washington Rd., Milwaukee, Wis.
ALLAN, W. D. M., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
ALLEN, H. S., Lock Joint Pipe Co., Kansas City, Mo.
*ALLEN, LESLIE H., Hawthorne Roofing Tile Co., 644 Main St., Glen Ellyn, Ill.
*ALLEN, O. T., American Steel & Wire Co., 7422 Bennett Ave., Chicago, Ill.
*ALLISON, LYMAN J., 115 S. Dearborn St., Chicago, Ill.
ALME, O., Leonard Construction Co., 1705 Ridge Ave., Evanston, Ill.
ALTER, FRED L., Manitowoc, Wis.
ANDERSON, A. F., 197 Downer Pl., Aurora, Ill.
*ARMSTRONG, W. D., Dearborn Brass Co., Cedar Rapids, Iowa.
*ARNOLD, M. A., Arnold Stone, Brick & Tile Co., Jacksonville, Fla.
*ANDEREGG, F. O., Mellon Institute, Pittsburgh, Pa.
ANDERSON, HARRY A., Crown Sidewalk & Block Co., Minneapolis, Minn.
*ANDERSON, LOUIS, JR., Alpha Portland Cement Co., Easton, Pa.
ANSEL, ANTON, 1115 Wisconsin Ave., Oak Park, Ill.
ARTHUR, ROBERT S., Dravo Equipment Co., New York, N. Y.
*ASHTON, ERNEST, Lehigh Portland Cement Co., Allentown, Pa.
*BARTRAM, GEO. C., 847 Ellicott Sq., Buffalo, N. Y.
*BATES, P. H., Bureau of Standards, Washington, D. C.
*BAYLIS, JOHN R., 6938 Crondon Ave., Chicago, Ill.
BAYSORE, C. R., Roos-Meyer-Hecht Co., Cincinnati, Ohio.
*BEGGS, NORMAN, Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
*BOGUE, R. H., Bureau of Standards, Washington, D. C.
BANNISTER, GEO. S., 115 S. Dearborn St., Chicago, Ill.
*BAKER, SAMUEL, 1215 Marion St., Scranton, Pa.
BAKER, H. H., Burrell Eng. & Constr. Co., La Grange, Ill.
BOYER, E. M., St. Joseph, Mo.
BOYNTON, G. H., Iowa Concrete Brick Co., Muscatine, Ill.
BABB, HOWARD J., Nelsen Concrete Culvert Co., Champaign, Ill.
*BEGGS, GEO. E., Princeton University, Princeton, N. J.
*BENHAM, SANFORD W., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
BENNETT, W. C., Chicago, Ill.
*BENNETT, W. M., Lehigh Portland Cement Co., Chicago, Ill.
*BENSON, NEWTON, 36 Burrington St., Providence, R. I.
*BENT, ERNEST F., 419 Grosse Bldg., Los Angeles, Calif.
BENT, F. J., Crystal Bldg., 166 W. Jackson Blvd., Chicago, Ill.
BERNHAM, F. M., 6248 Blackstone Ave., Chicago, Ill.
*BERNIER, NAPOLEON M., California Stucco Co., 411 Walden St., Cambridge, Mass.
*BERNT, HANS E., Universal Portland Cement Co., Chicago, Ill.
*BIGLER, H. P., Rail Steel Bar Assn., 111 W. Jackson Blvd., Chicago, Ill.

BISBEE, B. H., 175 Jackson Blvd., Chicago, Ill.
*BISCHOFF, J. M., 5915 Canton Ave., Detroit, Mich.
BJOINDAHL, RICHARD, 110 18th St., Moline, Ill.
BLAINE, ETHEL E., American Concrete Institute, Detroit, Mich.
BLOWERS, GEO. I., Bennett Mfg. Co., Chicago, Ill.
*BOGART, C. VAN DE, The Economy Concrete Co., Box 1857, New Haven, Conn.
BOLLENBACHER, J. C., 332 S. LaSalle St., Chicago, Ill.
*BOSCH, JACOB, 400 W. 107th St., Chicago, Ill.
*BOURNE, C. L., Consolidated Cement Corp., 1502 Greenwood Ave., Jackson, Mich.
*BOYER, E. D., 25 Broadway, New York, N. Y.
*BOYD, D. KNICKERBACKER, 112 S. 16th St., Philadelphia, Pa.
*BRADBYER, M. W., Besser Sales Co., 342 Monadnock Bldg., Chicago, Ill.
*BRAGGER, E. Y., The Sandusky Cement Co., Cleveland, Ohio.
*BRANDON, M. M., Underwriters Laboratories, 207 E. Ohio St., Chicago, Ill.
BREWER, G. D., 1058 E. Clinton St., Frankfort, Ind.
*BRICKETT, EDWARD M., Lehigh Portland Cement Co., Allentown, Pa.
BREWER, R. D., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
BRITT, GERALD F., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
*BROCK, A. S., Portland Cement Assn., 33 Grand Ave., Chicago, Ill.
BROCKHAUSEN, C. E., 140 S. Dearborn St., Chicago, Ill.
*BROKER, A. E., Badger Cement Tile Co., Plymouth, Wis.
BROOKMAN, LOUIS, 139 N. Clark St., Chicago, Ill.
*BROGA, GEO. F., 1218 W. Onondaga St., Syracuse, N. Y.
*BROWN, H. E., Consumers Supply Co., 42nd and State Sts., Milwaukee, Wis.
*BROWN, F. E., Holabird & Roche, Chicago, Ill.
BROWN, J. F., 3426 E. 89th St., Chicago, Ill.
*BROWN, REX L., 503 W. Main St., Urbana, Ill.
*BRUCE, WARREN C., 306 Planters Bldg., St. Louis, Mo.
BRUNS, H. F., Lt. Comdr., C. E. C., U. S. Navy, Great Lakes, Ill.
BUCHANAN, H. H., JR., 1280 Wilshire, Chicago, Ill.
*BUENTE, C. F., Concrete Products Co. of America, Pittsburgh, Pa.
*BULLEN, C. H., Concrete Pipe Co., Portland, Ore.
*BULLEN, CARROL A., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
BURGESS, S. W., 933 Leader Bldg., Cleveland, Ohio.
*BURKS, I. E., Tallassee Power Co., Calderwood, Tenn.
*BURNS, ALBERT S., 402 City Hall, Indianapolis, Ind.
*BURT, H. J., 1400 Monroe Bldg., Chicago, Ill.
BUSER, NATHANIEL E., Mount Morris, Ill.
*BUTLER, C. M., Marquette Cement Mfg. Co., LaSalle, Ill.
CALDWELL, EARL L., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
*CALDWELL, F. E., 99 Cortland St., Springfield, Mass.
CALIFF, E. J., Huron Portland Cement Co., Milwaukee, Wis.
CAMPBELL, JOHN R., Portland Cement Assn., Chicago, Ill.
*CAMPBELL, S. A., 251 Elmdorf Ave., Rochester, N. Y.
CAMPBELL, R. D., 310 McGee Bldg., Pittsburgh, Pa.
CAPOUCH, M. E., American Steel & Wire Co., 208 S. LaSalle St., Chicago, Ill.
*CARNOCHAN, D. E., National Lime Assoc., Minneapolis, Minn.
CARR, CHARLES A., D. H. Burnham & Co., 1900 Burnham Bldg., Chicago, Ill.
CARR, JAMES G., 7104 Sheridan Rd., Chicago, Ill.
*CARREL, F. G., Calumet Steel Co., 208 S. LaSalle St., Chicago, Ill.
*CARSWELL, J. R., Canadian Pacific Bldg., Toronto, Can.
*CASE, C. A., 32 N. Kostner Ave., Chicago, Ill.
*CATHCART, J., 410 N. Michigan Ave., Chicago, Ill.
CHADWICK, GEO. R., 5726 Winthrop Ave., Chicago, Ill.
*CONRADES, O. S., St. Louis Material & Supply Co., St. Louis, Mo.
*CHAPMAN, CLOYD M., 105 W. 40th St., New York, N. Y.
CHAMBERS, L. H., Chambers Constr. Co., Lincoln, Neb.

*CHANDLER, GEO. D., 2726 Whitney St., Detroit, Mich.
*CHANDLER, H. W., Pres. River Road Sand & Gravel Co., Merchantville, N. J.
CHANDLER, WARREN J., Lehigh Portland Cement Co., Allentown, Pa.
CHATTEN, MELVILLE C., 160 N. LaSalle St., Chicago, Ill.
CHILL, CHAS. M., Detroit Edison Co., 7411 Wykes Ave., Detroit, Mich.
*CHITTENDON, HOWARD L., Clinton, Conn.
*CHRISTENSEN, EINAR, 1600 Arch St., Philadelphia, Pa.
*CHUBB, JOS. H., 210 S. LaSalle St., Chicago, Ill.
*CLARK, A. P., Kalman Steel Co., 700 Forest Ave., Evanston, Ill.
*CLEARY, JAMES A., 201 N. West St., Indianapolis, Ind.
*CLEMMER, H. F., 40 Rector St., New York, N. Y.
*CLEVE, ALBERT, Portland Cement Assn., 221 Belleplaine Ave., Park Ridge, Ill.
*COCHRANE, W. C., Detroit Brass & Metal Works, Detroit, Mich.
*COHEN, A. BURTON, 1 Madison Ave., New York, N. Y.
*CLOUSING, LOUIS, 3441 Oakland Ave., Minneapolis, Minn.
CROMBIE, ROBERT A., 10715 Seeley Ave., Chicago, Ill.
CROSBY, E. S., Celite Co., Maplewood, N. J.
*CROSS, HARDY, Univ. of Illinois, Urbana, Ill.
*CRUM, R. W., Ames, Iowa.
CRUMP, HORACE E., Board of Comm. of Port of N. O., New Orleans, La.
CUPP, D. E., 925 8th Ave., New Brighton, Pa.
*CURTIS, A. J. R., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
*COLBURN, D. S., Marquette Cement Mfg. Co., 140 Dearborn St., Chicago, Ill.
*COLLINGS, O. ARTHUR, W. A. Collings Co., Santa Monica, Calif.
*COLLINGS, W. A., Mgr. W. A. Collings Co., Santa Monica, Calif.
*COLLINS, D. R., Concrete Products Association, Milwaukee, Wis.
*COLLINS, H. G., Chicago, Ill.
*COLMAR, DANIEL, Ramloc Stone Co., 22 Elk St., Albany, N. Y.
*CONDRON, T. L., Condron & Post, 53 W. Jackson Blvd., Chicago, Ill.
*COOK, F. W., 22nd and Kenneth Ave., Chicago, Ill.
*COOKE, C. E., Kalman Floor Co., 410 N. Michigan Ave., Chicago, Ill.
*COOPER, GILBERT, Ideal Concrete Co., Joliet, Ill.
*COPELAND, J. W., Central Tube Co., Minneapolis, Minn.
*CORBIN, M. ARRONET, Chicago Heights, Chicago, Ill.
CORCORAN, L. P., Portland Cement Co., 33 W. Grand Ave., Chicago, Ill.
*CRABBS, AUSTIN, The Cement Products Co., Davenport, Iowa.
*CRAIG, H. A., Universal Portland Cement Co., Winnetka, Ill.
*CRANE, THEODORE, Winchester Hall, New Haven, Conn.
DALE, F. E., 1109 Pennsylvania Bldg., Philadelphia, Pa.
DAVIS, E. E., 2244 Calumet Ave., Chicago, Ill.
DAYMUDE, C. A., 3509 Conville Ave., Detroit, Mich.
*DEINBOLL, F. K., 1263 Brockley Ave., Cleveland, Ohio.
DEMONEY, F. O., 5 N. LaSalle St., Chicago, Ill.
DERLETH, CHAS. P., 4030 Chouteau, St. Louis, Mo.
DEVER, CLARE, Huron Portland Cement Co., Detroit, Mich.
*DEVOS, A. W., Wm. H. Devos Co., Milwaukee, Wis.
*DICKERSON, OLIVER H., 510 Wolvin Bldg., Duluth, Minn.
*DIENHART, E. W., Cement City, Mich.
DOANE, L. H., 1315 Walnut St., Philadelphia, Pa.
*DOUTHETT, C. L., Waterloo Concrete Corp., Waterloo, Iowa.
DRAFFIN, JASPER O., University of Illinois, Urbana, Ill.
DRAINIE, JOHN G., Weary & Alford, 6748 Cornell Ave., Chicago, Ill.
*DREISBACH, EDW. E., Nazareth Cement Co., Nazareth, Pa.
DRESSER, E. M., Arlington, Minn.
DUGAN, CHAS. B., 1118 Straus Bldg., Chicago, Ill.
*DURGIN, F. L., JR., Cramp & Co., 801 Denckla Bldg., Philadelphia, Pa.
*DUTTON, C. B., Besser Sales Co., 53 W. Jackson Blvd., Chicago, Ill.

EACH, CHAS. H., Illinois Steel Co., 3426 E. 89th St., Chicago, Ill.
*EADES, HARRY H., National Steel Fabric Co., Pittsburgh, Pa.
*EARLEY, JOHN J., 2131 G St., N. W., Washington, D. C.
*EASTERDAY, PHIL, The Phil Easterday Co., Terre Haute, Ind.
*EASTERDAY, HORACE, The Phil Easterday Co., Terre Haute, Ind.
*ECKDALE, W. P., Universal Portland Cement Co., 7322 Ekerhart Ave., Chicago, Ill.
*ECKERT, R. T., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
*EDDY, W. PAUL, 102 N. Beech St., Syracuse, N. Y.
EDLUND, LAWRENCE L., Gardner & Lindberg, 140 S. Dearborn St., Chicago, Ill.
*EDWARDS, C. E., 331 Mich. Trust Bldg., Grand Rapids, Mich.
*EGE, C. R., 33 W. Grand Ave., Chicago, Ill.
*EGELHOFF, R. F., 11 Goodell St., Buffalo, N. Y.
*EITZEN, HENRY R., Kalman Steel Co., Philadelphia, Pa.
ELSHBAIN, L., Smith & Brown, Chicago, Ill.
*ELWELL, J. S., 927 15th St., N. W., Washington, D. C.
*EMERSON, H. B., 318 Broadway, Methuen, Mass.
EMDING, RALPH W., Chicago, Ill.
*EPPES, N. A., Gulf Concrete Pipe Co., Houston, Tex.
ERICKSON, J. A., 531 Dorothy Lane, Chicago, Ill.
*ERICKSON, RALPH E., 780 Dearborn St., Chicago, Ill.
*ESCHENBRENNER, H., Universal Concrete Products Co., Martinsville, W. Va.
*EVERHART, C. C., 6038 Drexel Ave., Chicago, Ill.
EVERROLE, H. E., Universal Portland Cement Co., Universal, Pa.
*EY, VICTOR, 57th and Burnside Ave., Woodside, L. I., N. Y.
*FAIRCHILD, L. F., 3195 Lake Ave., Rochester, N. Y.
*FARMER, HOMER G., 526 Frick Bldg., Pittsburgh, Pa.
FARREN, J. M., 4446 N. Canfield, Chicago, Ill.
*FATOUT, H. D., H. J. Baker & Co., Indianapolis, Ind.
*FATOUT, RAY T., Hall Constr. Co., 406 Board of Trade, Indianapolis, Ind.
*FALCO, LOUIS A., The Decorative Stone Co., New Haven, Conn.
FELLOWS, C. H., The Detroit Edison Co., Detroit, Mich.
*FERGE, HARRY, Kohler, Wis.
*FERGUSON, M. W., Roanoke, Va.
*FLAM, STEPHEN, Supertile Machinery Corp., Tecumseh, Mich.
*FOLEY, R. A., Superior Products Co., Detroit, Mich.
FORD, C. L., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
*FORREST, V. E., Minneapolis, Minn.
FORSTE, HERMAN W., 3303 Orion Ave., Cincinnati, Ohio.
FORTIN, J. T., 600 Blue Island Ave., Chicago, Ill.
*FOSDICK, C. E., Dugan Concrete Co., 5263 Eastern Ave., Cincinnati, Ohio.
FOSHINBAUR, V. G., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
*FOSTER, ALEXANDER, 6136 Oxford St., Philadelphia, Pa.
FOSTER, C. WILLARD, Foster Eng. Service Co., Indianapolis, Ind.
*FOSTER, C. B., 726 K. of P. Bldg., Indianapolis, Ind.
*FOURNIE, A. J., Swansea Stone Wks., Belville, Ill.
FOX, JOHN J., 38 S. Dearborn St., Chicago, Ill.
*FRANCIS, CARL, Phoenix Portland Cement Co., 1500 Chestnut St., Philadelphia, Pa.
FRANKLIN, J., Anchor Concrete Machy. Co., Chicago, Ill.
*FRAUENFELDER, HERMAN, 4600 Chippewa St., St. Louis, Mo.
*FREEMAN, J. E., 122 S. Michigan Ave., Chicago, Ill.
*FREEMAN, P. J., 519 Smithfield, Pittsburgh, Pa.
FREIBERG, FREDERICK A., 6809 Cornell Ave., Chicago, Ill.
*FULCHER, L. H., Jackson, Miss.
*FULLENWIDER, C. V. R., The Philip Carey Company, Cincinnati, Ohio.
*GARBARINI, R. J., Cadillac Cast Stone Co., 4469 Seyburn Ave., Detroit, Mich.

*GARDINER, LION, Lakewood Engineering Co., Cleveland, Ohio.
GASTON, L. S., Holabird & Roche, 6048 Woodlawn, Chicago, Ill.
GEMENY, BLAINE B., 7632 Cottage Grove Ave., Chicago, Ill.
GEBBART, B. R., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
GERHARDT, PAUL, JR., 64 W. Randolph St., Chicago, Ill.
GIBSON, W. W., 308 National Bldg., Milwaukee, Wis.
GISH, H. J., Stewart Sand Co., Kansas City, Mo.
GISH, R. R., 6048 Woodlawn Ave., Chicago, Ill.
*GILKEY, HERBERT J., University of Colorado, Boulder, Colo.
GIRYOTAS, W. J., 136 Central Ave., Chicago, Ill.
GLADDEN, A. H., 2904 Grand Ave., Apt. 105, Milwaukee, Wis.
GLASNER, J. T., Aurora, Ill.
GOCHMAUER, C. O., 1101 N. Leader, Appleton, Wis.
GOES, E. J., 1063 46th St., Milwaukee, Wis.
*GOLDBECK, A. T., National Crushed Stone Assn., 651 Earle Bldg., Washington, D. C.
*GONNERMAN, H. F., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
GOODWIN, R. A., 542 Monadnock Bldg., Chicago, Ill.
GOSSWEIN, O. H., Universal Portland Cement Co., Chicago, Ill.
GOULD, HARRY, 4024 West End Ave., Chicago, Ill.
GRAHAM, R. J., 30 N. Michigan Ave., Chicago, Ill.
GRANGER, ALFRED, 332 S. LaSalle St., Chicago, Ill.
*GRAVELL, WM. H., 225 S. 15th St., Philadelphia, Pa.
*GRAY, H. A., 71 Wallace St., Somerville, Mass.
*GRIFFIN, H. E., 53 W. Jackson Blvd., Chicago, Ill.
*GRIFFITH, E. A., 410 Hampton Ave., Wilkinsburg, Pa.
*GROTH, E. G., Moline, Ill.
*GROTHE, WM. F., Osage Indian Agency, Pawhuska, Okla.
GRUBE, L. E., Sheboygan, Wis.
*GRUTSCH, FRANK J., 957 N. and S. Road, St. Louis, Mo.
*HAGENER, ARTHUR, Union Bldg., Cleveland, Ohio.
HAIGHT, E. E., 139 N. Clark St., Chicago, Ill.
HALL, EMERY STANFORD, 310 S. Oakwood Ave., Highland Park, Mich.
HALLINGER, JOHN, 3323 N. Clark St., Chicago, Ill.
*HANSARD, O. H., State Highway Dept., Nashville, Tenn.
HARDESTY, H. W., The Rein-Hast Conc. Block Co., Ft. Thomas, Ky.
*HARDING, E., Ferro Concrete Constr. Co., Cincinnati, Ohio.
HARKNESS, JOHN C., 2539 Arlington Ave., Chicago, Ill.
*HARRIS, WALLACE R., Le Roy, N. Y.
*HARRISON, MERRITT, Harrison & Turnock, 500 Board of Trade, Indianapolis, Ind.
*HART, W. E., 33 W. Grand Ave., Chicago, Ill.
*HARTLESS, T. B., Froehling & Robertson, Richmond, Va.
*HARTMAN, ROBERT, Alpha Portland Cement Co., 140 S. Dearborn, Chicago, Ill.
HATT, K. A., 20th and Prairie Ave., Chicago, Ill.
*HATT, W. K., Purdue University, Lafayette, Ind.
HAUPT, E., Strobel Steel Construction Co., 53 W. Jackson, Chicago, Ill.
*HAVLIK, ROBT. F., 561 Garfield Ave., Aurora, Ill.
HAWKINS, E. R., Holabird & Roche, 7028 Creager Ave., Chicago, Ill.
*HAYDE, S. J., 403 Mutual Bldg., Kansas City, Mo.
*HAYDEN, J. E., Crystalite Reduction Co., Iron Mountain, Mich.
HAYNES, CHAS. W., Tribune Tower, Chicago, Ill.
*HECK, HERMAN H., Louisville Hydro Elec. Co., Louisville, Ky.
HEEB, E., Standard Concrete Pipe & Curb Co., New Orleans, La.
HEDBLADE, ERNEST E., 1030 Duncan Ave., Elgin, Ill.
HESSLER, H. C., 1328 Amy Ave., Whiting, Ind.
*HERBST, H. W., Serviced Products Corp., 6051 W. 65th St., Chicago, Ill.
HEJER, GEO., Hoyer-Ellefsen, Oslo, Norway.

*HERZOG, C. J., 310 Magee Bldg., Pittsburgh, Pa.
*HEWES, GEO. C., 235 Geneva St., Decatur, Ga.
*HILKER, E. W., 16th and State St., Granite City, Ill.
*HIRSCHBERG, W. P., Milwaukee, Wis.
*HODNETT, RALPH M., Clarendon Hills, Ill.
HOCKE, JOHN, 1604 E. 79th St., Chicago, Ill.
HOLINGER, ARNOLD C., 545 Brompton Ave., Chicago, Ill.
*HOLM, W. M., 1018 W. State St., Chicago, Ill.
*HOWARD, W. F., 2048 Superior St., Chicago, Ill.
*HOUGH, J. F., 33 W. Grand Ave., Chicago, Ill.
*HOVDE, A. D., St. Paul, Minn.
*HOWE, HENRY L., Eng. Dept. City of Rochester, Rochester, N. Y.
*HOWE, H. N., Gardner & Howe, Memphis, Tenn.
HUBBARD, A. H., 38 S. Dearborn St., Chicago, Ill.
HUBBARD, BERT C., 9911 S. Robey St., Chicago, Ill.
*HUBBARD, FRED, Youngstown, Ohio.
*HUGHES, E. E., Franklin Steel Works, Franklin, Pa.
*HUMPHREY, RICHARD L., Philadelphia, Pa.
*HUNT, PHILIP L., Hunt Stone-Tile Co., Salem, Mass.
HOUSE, FRANK J., 2624 Thayer St., Evanston, Ill.
HUSPEK, L. L., 5518 Kunbark Ave., Chicago, Ill.
*HUTCHINS, T. A., Wolverine Portland Cement Co., Coldwater, Mich.
*HUXHOLD, P. F., 1125 Garfield St., Oak Park, Ill.
*HYNES, W. J., 858 Dupont St., Toronto, Can.
*INGEMANSON, T. W., 5944 W. Erie St., Chicago, Ill.
*IRWIN, A. C., 141 S. Waiala Ave., La Grange, Ill.
JABIN, FRED W., Kalman Floor Co., Chicago, Ill.
*JACKSON, FRANK H., U. S. Bureau of Public Roads, Washington, D. C.
JACOBS, A. P., 500 Camp St., New Orleans, La.
JACOBSON, H. E., 210 S. LaSalle St., Chicago, Ill.
JELINEK, BENJ., Milwaukee, Wis.
JENKINS, WM. B., The Billings-Chapin Co., Cleveland, Ohio.
JELLSON, BYRON H., 6 N. Michigan Ave., Chicago, Ill.
JOHANNES, GUST H., 915 Olive St., St. Louis, Mo.
*JOHNSON, A. V., Ideal Cement Stone Co., Omaha, Neb.
JOHNSON, P. O., Anchor Stone Co., 2937 Lundale Ave., Minneapolis, Minn.
JOHNSON, J. KENNY, 4309 N. Paulina, Chicago, Ill.
*JOHNSON, C. S., Champaign, Ill.
JOHNSON, L. E., 508 Home Bank Bldg., Chicago, Ill.
*JOHNSON, NATHAN C., 260 Glenwood Road, Englewood, N. J.
JOHNSON, O. A., R. W. Hunt Co., Chicago, Ill.
*JOHNSON, ROBERT C., Fond du Lac, Wis.
*JOHNSON, H. E., Box 1765, Houston, Tex.
JOHNSON, R. W., 2112 Tribune Tower, Chicago, Ill.
*JOHNSON, T. H., Sioux City, Iowa.
*JOHNSON, VIRGIL L., 29 W. Upsal St., Philadelphia, Pa.
*JOHNSON, WM. R., 519 Ford St., Geneva, Ill.
*JOHNSTON, HUGH H., Watertown, S. D.
KAISER, W. G., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
*KAMIENSKY, ALBERT J., 129 Eaton St., Buffalo, N. Y.
KANZIG, C. S., 908 Chamber of Commerce Bldg., Chicago, Ill.
KAYSEN, T., JR., The McKays Co., St. Paul, Minn.
*KEAN, R. E., 300 Transportation Bldg., Detroit, Mich.
*KEARNEY, JAMES C., Western Waterproofing Co., Detroit, Mich.
KEATINGS, PAUL F., The Atlas Portland Cement Co., Chicago, Ill.
KEITH, F. W., 550 Melrose St., Chicago, Ill.
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